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Partially Composite Concrete Sandwich Panels

by

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in

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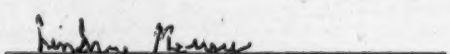

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Abstract

The primary objective of this research was to develop behaviour models for the design of partially composite concrete sandwich panels. A series of flexural tests were performed to aid in the development of this objective. In addition to the flexural test it was also necessary to perform ancillary tests to characterize the behaviour of the individual sandwich panel components. The results of the ancillary tests were used in the behaviour models to predict the results of the flexural tests.

The behaviour of the flexural test specimens was comparable to the behaviour of coupled shear walls. The concrete wythes behave similar to the shear walls while the wythe connectors behave similar to the coupling beams. Design and analysis procedures similar to those used for coupled shear walls were developed for concrete sandwich panels. The procedures are presented in this thesis.

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List of Symbols

a = depth of equivalent rectangular stress block

A = cross sectional area

A_g = gross area of concrete section

A_p = area of prestressing steel

A_{strand} = area of an individual prestressing strand

A_t = transformed area

b = sandwich panel width

c = distance from extreme compression fibre to neutral axis

c_1 = distance between 'compression face' and neutral axis

c_2 = distance between 'tension face' and neutral axis

C = compressive force

d_p = distance from extreme compression fibre to centroid of prestressing steel

d_t = depth of uncracked concrete in tension

e = eccentricity of the prestressing force parallel to axis measured from the centroid of the section

e_1 = distance between centroid of uncracked section and the centre of gravity of the stress diagram between $\varepsilon = 0$ and $\varepsilon = \varepsilon_1$

e_2 = distance between centroid of uncracked section and the centre of gravity of the stress diagram between $\varepsilon = 0$ and $\varepsilon = \varepsilon_2$

E_c = modulus of elasticity of concrete

E_p = modulus of elasticity of prestressing strand

f_c = stress experience by concrete due to design loads

f'_c = compressive strength of concrete

$f_{c(max)}$ = maximum concrete stress in compression zone

$f_{c1(avg)}$ = average compressive stress between 'compression face' and neutral axis

$f_{c2(avg)}$ = average compressive stress between 'tension face' and neutral axis

f_{ce} = stress in concrete at the level of prestressing steel after all losses

f_{cp} = compressive stress at depth of prestressing strand

f_{pe} = effective stress in prestressing steel after allowance for all prestress losses

f_{pr} = stress in prestressing strand when member is at ultimate resistance

f_{pu} = tensile strength of prestressing strand

$f_{p/s}$ = average prestress force in concrete

f_y = yield strength of reinforcing steel

h_{comp} = thickness of compression wythe

h_{insul} = thickness of insulation

h_{tens} = thickness of tension wythe

I = moment of inertia of cross section

jd = moment arm

k_p = coefficient used in calculating f_{pr}

L = unsupported length of sandwich panel

M = internal resisting moment

M_{avg} = average moment resistance of the concrete wythes

M_1 = moment resistance of the compression wythe

M_2 = moment resistance of the tension wythe

M_{c1} = resulting moment of P_{c1} about the centroid of the uncracked section

M_{c2} = resulting moment of P_{c2} about the centroid of the uncracked section

M_{cr} = cracking moment

M_f = factored moment due to design loads

M_n = nominal moment resistance of section

M_p = resulting moment of P_p about the centroid of the uncracked section

M_r = factored moment resistance of section

M_{sum} = summation of moments

M_t = resulting moment of P_t about the centroid of the uncracked section

M_x = transverse moment due to stripping of panel

M_z = longitudinal moment due to stripping of panel

n = modular ratio

N = normal force

P_{c1} = average compressive force between 'compression face' and neutral axis

P_{c2} = average compressive force between 'tension face' and neutral axis

P_e = prestress force after all losses

P_p = force in the prestressing strand

P_{sum} = summation of forces

P_t = tensile force carried by the concrete

S = section modulus

T = tensile force

T_1 = tensile force in prestressing strands in compression wythe assuming a fully-composite panel

T_2 = tensile force in prestressing strands in tension wythe assuming a fully-composite panel

V = internal resisting shear

V_{insul} = shear transferred through the insulation between locations of maximum moment and zero moment

V_{sc} = load carried by shear connectors between locations of maximum moment and zero moment

w = specified load per unit length or specified load per unit area

w_f = factored design load per unit length or factored design load per unit area

w_n = nominal load per unit length or nominal load per unit area resisted by the sandwich panel

w_r = factored load per unit length or factored load per unit area resisted by the sandwich panel

x = estimated end slip

x_1 = distance from the 'compression face' to the strand in the compression wythe

x_2 = distance from the 'tension face' to the strand in the tension wythe

y_t = distance between centroid of uncracked section to extreme tension fibre

α_1 = ratio of average stress in rectangular compression stress block to specified strength of concrete

β_1 = ratio of depth of rectangular compression stress block to distance from extreme compression fibre to neutral axis

δ = variable used in Moment – Area theorm

= deflection or displacement

ε_0 = concrete strain corresponding to peak concrete stress on stress – strain curve

ε_1 = strain in concrete at extreme compression fibre

ε_2 = strain in concrete at extreme tension fibre

ε_{ce} = the strain in concrete corresponding to a stress of f_{pe} in the prestressing steel

ε_p = strain in concrete at level of prestressing steel

ε_{pe} = strain in prestressing steel after allowances for all prestress losses

ε_{ps} = $\varepsilon_{pe} + \varepsilon_{ce} + \varepsilon_p$

λ = correction factor to account for low density concrete

μ = coefficient of friction

μ_f = factored coefficient of friction

ρ = radius of curvature

ϕ = curvature

ϕ_c = resistance factor for concrete

ϕ_p = resistance factor for prestressing strand

ϕ_s = resistance factor for reinforcing steel

ϕ_{sc} = resistance factor for shear connectors

ϕ_{insul} = resistance factor for coefficient of friction between insulation and concrete

θ = inclination angle from horizontal

1 Introduction

1.1 Description of Sandwich Panels

The purpose of the research project is to improve the structural design of insulated precast wall panels. These panels, also known as sandwich panels, consist of a layer of insulation sandwiched between inner and outer layers (wythes) of concrete that are held together by connectors (Figure 1.1). Applications of sandwich panels include warehouse type buildings, movie theatres, office buildings, and various industrial buildings. Sandwich panels offer rapid construction of tall walls, good sound insulation, improved thermal insulation and energy efficiency, and good durability and fire resistance. Precasting the sandwich panels in a factory allows the production of a variety of architectural finishes. Various finishes include; a smooth surface, a sandblasted finish, an exposed aggregate finish achieved with a retardant, a ribbed surface, or a surface with reveals.

Canada is a world leader in the design and construction of precast concrete sandwich panels because the rapid construction procedures suit the short outdoor construction season. Sandwich panels are fabricated indoors using efficient automated manufacturing processes. The panels can be readily erected on site throughout a Canadian winter thus speeding construction and extending the construction season. Erected sandwich panels also allow the building to be closed in quickly providing improved working conditions for all trades.

In addition to insulating the building, sandwich panels must also transfer loads. Many precast sandwich panels are strictly cladding panels designed to resist transverse loads only. Other types include load-bearing wall panels designed to resist transverse and axial loads, as well as shear walls designed to resist in-plane shear. This thesis only considers sandwich panels designed as cladding panels.

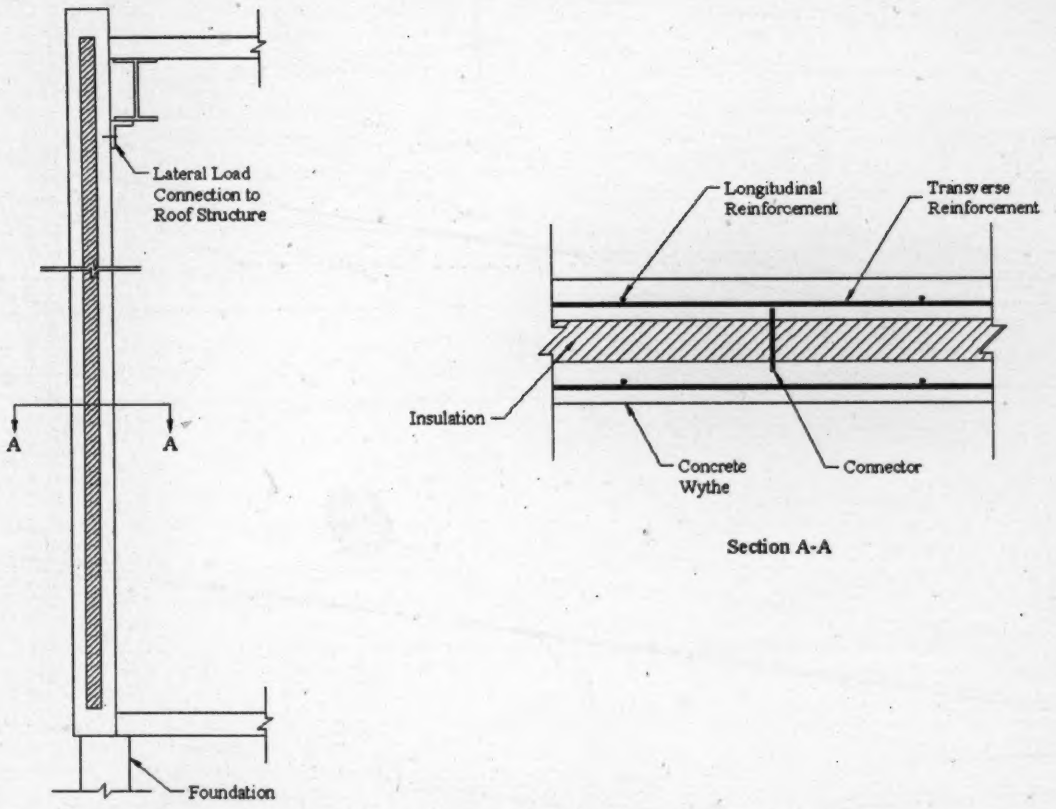

Figure 1.1 Components of Sandwich Panels

1.2 Structural Behaviour of Sandwich Panels

The structural performance of sandwich panels, in terms of strength and stiffness primarily depends upon how the concrete wythes are connected to each other through the insulation layer. Shear is transferred through the connectors used to tie the two concrete wythes together resulting in a composite or partially composite behaviour. Various styles of connectors are presented in the PCI Sandwich Panel Committee Report (PCI, 1997).

Sandwich panels are designed and detailed to be non-composite, composite or partially composite panels. Figure 1.2 shows the difference in behaviour between the three sandwich panel types. Non-composite panels are designed and detailed using flexible connectors, referred to as "tension or compression ties", that minimize the shear transfer between the two concrete wythes. In doing so, the concrete wythes act virtually independent of one another and carry loads based on their relative stiffness. Composite

panels are designed and detailed using stiff connectors, referred to as "shear connectors", which are capable of transferring shear between the two concrete wythes. The shear transfer allows the two concrete wythes to work together to resist loads. A fully composite panel would have enough shear transfer to cause the two wythes to act as one solid unit. That is, "plane sections remain plane". The behaviour of partially composite panels is somewhere between the behaviour of non-composite panels and composite panels. Various definitions are presented in the literature to describe the degree of composite action. There is no consensus in the literature on a design method for partially composite sandwich panels.

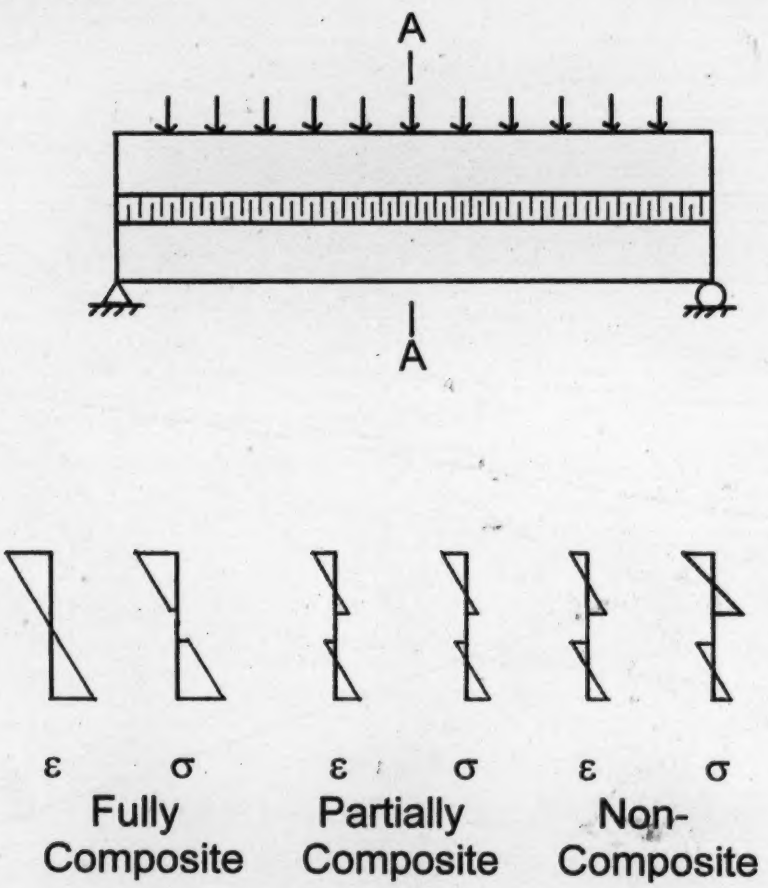

Figure 1.2 Stress and Strain Profiles at Section A-A for Various Degrees of Composite Action

1.3 Research Objective

The primary research objective was to improve the structural design of precast concrete sandwich panels used for cladding. The development of a functional analysis model to predict the deformation and the load carrying capacity of sandwich panels is a key output of this research. Tests of full size wall panels are used to confirm that the analysis model provides reasonable predictions.

1.4 Thesis Organization

A literature review was performed. The purpose of the literature review was to identify the important structural considerations for sandwich panels. In addition, literature was reviewed to examine the experimental program and analysis from previous research projects to aid in the development of this research project. The literature review is presented in Chapter 2.

After completion of the literature review, a preliminary experimental program was performed. The preliminary experimental program involved flexural tests along with ancillary tests performed on specimens obtained from existing wall panels. The purpose of the preliminary experimental program was to gain a better understanding of the flexural behaviour of concrete sandwich panels prior to designing and casting full-scale test panels. Chapter 3 provides a description of the test procedures and test specimens along with the test results for the preliminary flexural tests and ancillary tests. The theory for the analysis of the preliminary test results is outlined in Chapter 4. The results of the analysis are also provided in this chapter.

From the preliminary experimental program it was apparent that the design of sandwich panels could be improved through the reduction of materials used to construct them. The knowledge obtained from the preliminary experimental program was used to design and build a full-scale flexural test panel. Further ancillary tests were also required. Chapter 5 provides a description of the test procedures and test specimens, the test results, and the analysis of the results for the full-scale flexural tests and ancillary tests.

The summary and conclusions are presented in Chapter 6. Also, detailed analysis and design calculations are provided in the Appendicies.

2 Literature Review

2.1 Introduction

The existing literature on sandwich panels was reviewed. References providing general background were used to identify the important structural considerations for sandwich panels. References presenting original experimental data were also reviewed. These references were examined to determine how the experimental program and analysis were performed, and the styles of connectors considered. Gaps in the knowledge base and research needs were identified. These research needs formed the basis for the experimental program that was performed.

2.2 General Background

Important considerations include: the advantages of composite and non-composite panels; thermal considerations; and the structural design and analysis procedures for sandwich panels.

2.2.1 Composite Action vs. Non-Composite Action

The reason for wanting composite action is the improved structural efficiency. That is, to create taller or stronger panels with less material. A non-composite panel is typically constructed with a thick structural wythe and a thin architectural wythe. The structural wythe is designed to resist all applied loads along with the weight of the architectural wythe. As a result the weight and thickness of a non-composite panel are greater than those of a composite panel. Figure 2.1 shows that with composite action one has a much larger lever arm for the tension / compression couple.

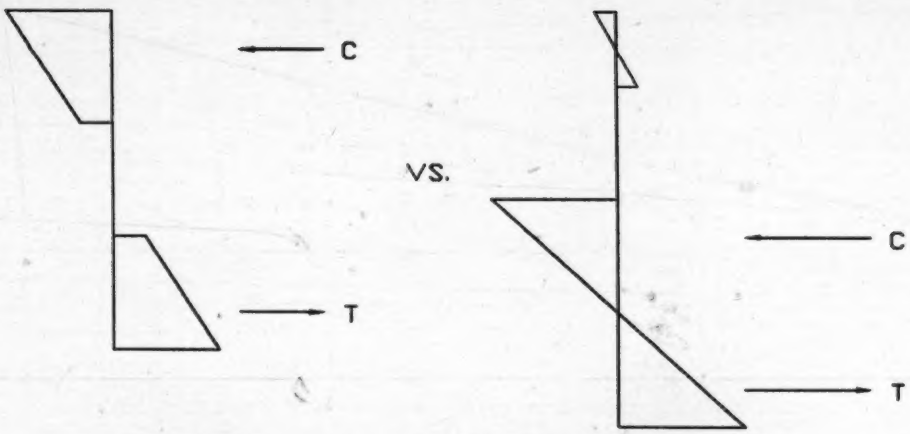

Figure 2.1 Composite Action vs. Non-Composite Action

One reason for demanding non-composite action is to reduce panel bowing (Figure 2.2). The PCI Sandwich Panel Committee Report (PCI, 1997) discusses the many factors that can contribute to panel bowing. Panel bowing may occur because of a large thermal gradient between the two concrete wythes. For example, when a structure is exposed to direct sunlight the exterior wythe heats up and expands relative to the interior wythe. In composite panels this thermal gradient causes the sandwich panel to bow outwards. This is less of a concern for non-composite panels because the structural wythe, located on the interior face of the panel, is insulated from the extreme temperature changes. Furthermore, the flexible connectors used in non-composite panels to hold the two concrete wythes together allow the exterior wythe to expand and contract without inducing stresses that could cause cracking. Suitable edge details are of course necessary to allow relative motion between the two wythes.

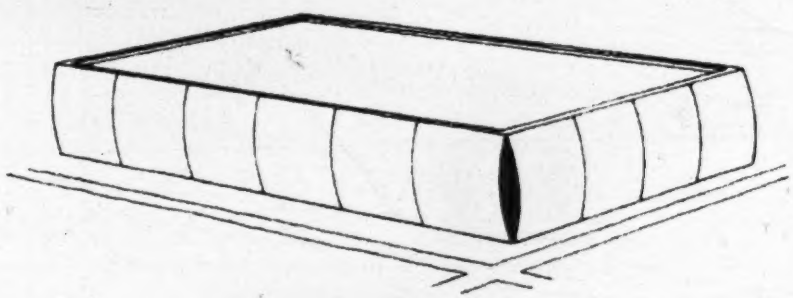

Figure 2.2 Panel Bowing

2.2.2 Thermal Considerations

Sandwich panels are insulated to minimize heating and cooling costs. Although thermal design is a separate issue, thermal design and structural design are somewhat interrelated. The type and thickness of insulation used may impact the structural performance. Different types of insulation are outlined in the PCI Sandwich Panel Committee Report (PCI, 1997). The structural design also impacts the thermal properties of the sandwich panels. There is a general desire to minimize the size and number of shear connectors to minimize thermal bridging. McCall discusses the effects of thermal bridging on the thermal performance of sandwich panels (McCall, 1985).

2.2.3 Existing Design and Analysis Methods

Design and analysis procedures for fully composite, and fully non-composite precast sandwich panels are presented in the PCI Sandwich Panel Committee Report (PCI, 1997). No procedures are presented for partially composite panels. It is suggested that partially composite panels should be designed and analysed as fully composite panels during stripping, handling, and erection. The initial composite action results from significant bond between the insulation and concrete that allows shear transfer between the two concrete wythes. For in-service loading it is suggested that partially composite panels be designed as non-composite panels. It is likely the bond between the insulation and concrete breaks down with time, therefore it is not relied upon for in-service loading and subsequent ultimate loading. The shear transfer that occurs through the wythe connectors is still present but completely ignored. Another design methodology commonly used for partially composite panels only considers the shear force transfered through the shear connectors and the corresponding couple. This design methodology ignores the bending resistance of each wythe and results in an overly conservative design.

Holmberg and Plem (1965) present solutions based on the theory of elasticity. Equations were derived for sandwich panels using a truss-like connector. Solutions account for axial loads, transverse loads, loads due to the effects of temperature and shrinkage, and combinations of the above. The equations become invalid when yielding or cracking occurs. Because most panels use other types of connectors and experience yielding and cracking, the elastic solutions have limited practical value.

The behaviour of sandwich panels is comparable to the behaviour of coupled shear walls. Figure 2.3 shows two walls coupled together by beams (MacGregor, 2000). Figure 2.3(a) uses hinged coupling beams; therefore no shear transfer occurs between the two walls. This coupled wall system is comparable to a non-composite sandwich panel in which the walls are equivalent to the concrete wythes and the hinged coupling beams are equivalent to the tension / compression ties. The total resisting moment of this system is:

$$M = M_{w1} + M_{w2} \qquad [2.1]$$

Figure 2.3(b) uses stiff coupling beams that transfer shear between the two walls. This coupled wall system is comparable to a composite or partially composite sandwich panel in which the walls are equivalent to the concrete wythes and the stiff coupling beams are equivalent to the shear connectors. The total resisting moment of this system is:

$$M = M_{w1} + M_{w2} + (T_o \times l) \qquad [2.2]$$

where: $T_o = \sum_{i=1}^{n} V_{bi}$ [2.3]

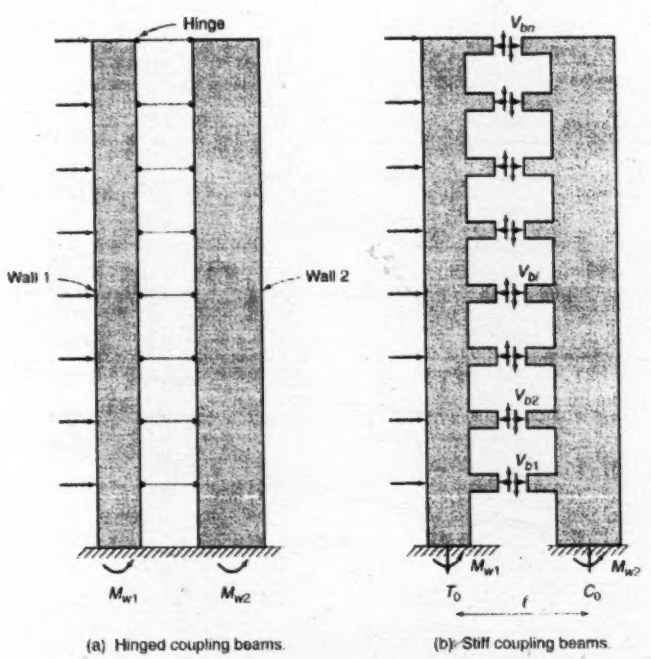

Figure 2.3 Coupled Shear Walls (MacGregor, 2000)

A key element of this research project is to characterize the V_{bi} term for certain types of shear connectors used in sandwich panel construction.

2.3 Existing Sandwich Panel Research

This section examines experimental studies to see how the experiments were performed, how the data were analysed, and how the panels behaved.

Einea (1992) investigates panels with proprietary non-metallic shear connectors. Traditionally concrete and/or steel components have been used for the shear connection. Concrete and steel connectors have a low thermal resistance, resulting in a thermal bridge that decreased the overall thermal resistance of the panel. To improve the thermal efficiency of the panel it is necessary to have an uninterrupted thermal barrier. This was achieved through the use of fibre reinforced plastic (FRP) connectors.

Einea's research included three testing phases followed by an analysis phase. The first two stages of testing involved "unit shear tests" on various styles of shear connectors. Tests were performed on specimens with and without a bond-breaker between the insulation and the concrete layers. The specimens containing a bond-breaker allow the actual characteristics of the shear connectors to be measured. During the third phase of testing two small-scale flexural tests were performed. The specimens were tested in a flat horizontal position, simply supported at both ends and were loaded with concentrated loads at third points. The first flexural specimen contained a bond-breaker between the insulation and the concrete while the second specimen did not.

Three analytical models were used in an attempt to predict the test results. The models include a differential equation solution and two finite element models. A linear material finite element model was developed to simulate the results from the unit shear tests and a non-linear finite element model was developed to simulate the flexural test results. The models predicted a much stiffer response than was observed during testing. The percentage of composite action was defined as the ratio between the actual ultimate strength and the ultimate strength calculated assuming a fully composite panel.

Culp (1994) performed tests on two full-scale specimens containing the FRP connector recommended by Einea. He also tested two full-scale specimens containing conventional steel truss connectors. The 30'-6" tall by 8' wide specimens were tested in a vertical position. The panels had simply supported boundary conditions at the top and bottom edges. A uniform horizontal pressure, simulating wind loads, was applied with airbags. The airbags reacted against a precast double-tee section and were confined along the edges to prevent the airbags from failing. The airbags were pressurized with an air tank and pump. Pressures were controlled with a regulator. The mid-span deflection of the specimens was measured. Strain gauges were also used to measure the strains in the connectors and in the concrete. During testing, the airflow was stopped while readings were taken. Readings were taken at ¼ " displacement increments.

The analysis of the test results was performed using the solution developed by Holmberg and Plem (1965). Culp defined the percentage of composite action using the following equation:

$$\%Composite = \frac{\left(S_{specimen} - S_{non-composite}\right)}{\left(S_{composite} - S_{non-composite}\right)} \qquad [2.4]$$

where S is the slope of the load vs. mid-span deflection curve. This value was then used to calculate an effective moment of inertia for the panels tested.

$$I_{effective} = I_{composite} \times \%Composite \qquad [2.5a]$$

It appears that there is an error in Eqn. [2.5a]. From Eqn. [2.4] one can derive the following equation:

$$I_{effective} = I_{non-composite} + \%Composite \times \left(I_{composite} - I_{non-composite}\right) \qquad [2.5b]$$

Stine (1992) performed flexural tests on two series of panels. The first test series were run-of-production panels produced for a typical commercial project. The production panels had solid concrete zones, which interrupted the insulation, located at the various stripping and handling inserts. Concrete ribs, typically 1/2" to 5/8" wide, were also formed in the gaps between the insulation sheets. The second test series was produced without solid concrete zones at the stripping and handling inserts and without concrete in the gaps between the insulation sheets. No bond-breaker was provided

between the insulation and the concrete. Unit shear tests were performed to determine the shear transfer characteristics of the steel truss shear connectors used and the insulation. One wall was also tested in fatigue.

The objective of the experimental program was to determine how the number, spacing and orientation of the steel truss shear connectors affected the degree of composite action obtained by the wall panels. The degree of composite action was defined as follows:

$$\%M_{com} = \frac{M_{ext} - (M_{tw} + M_{bw})}{M_{ext}} \times 100\% \qquad [2.6]$$

Where: $\%M_{com}$ = percent of composite moment
M_{ext} = external moment at mid-span of panel ($wl^2/8$)
$M_{tw,\,bw}$ = internal non-composite moment in the top or bottom wythe ($S \cdot E_c \cdot \varepsilon_{cr}$) assuming they are uncracked
S = section modulus of a single uncracked wythe
E_c = modulus of elasticity of the concrete
ε_{cr} = average strain difference at outer faces of the wythe, as determined from test data

The 8' wide by 16' long flexural specimens were tested in the horizontal position. The specimens were simply supported and subjected to a uniform, lateral load. To apply the uniform load an airbag was positioned under the test panel and confined on the sides by a frame constructed of steel decking. The ends of the airbag were confined using wide flange sections, which also acted as the end supports. The load was measured using an electronic pressure transducer and verified using a dial pressure gauge. To determine the relative mid-span deflection, wire potentiometers were located at the mid-span of the panel and linear potentiometers were located at the supports. In an attempt to obtain the strain gradient through the depth of the panel, strain gauges were placed on the prestressing strands, dummy bars located near the inside surface of both concrete wythes, and on the exterior concrete surfaces. Strain gauges were also used to measure the strains experienced by the shear connector truss diagonals.

The fatigue test was performed using a third point loading system as opposed to a uniform load. The specimen was cyclically loaded to simulate curvatures that would

occur due to thermal gradients. These values were obtained from an analysis modelling temperature differences between the top and bottom concrete wythes.

The production panel tests showed that most of the shear transfer between the two concrete wythes occurred through the solid concrete zones and through the concrete ribs between the insulation sheets. In the modified series where these were eliminated, shear was transferred between wythes through a combination of truss connectors and bonded insulation. The insulation increased the shear transfer between the wythes by about 50 to 65% of that carried by the truss connectors. Fatigue testing to 55,000 load cycles showed forces experienced by the truss diagonals gradually increased. This suggests gradual loss of bond between the insulation and the concrete.

Stine attempted to predict the truss diagonal forces. He used a strength of materials approach, a computer model using a frame analysis, and a finite element model. The above methods resulted in an overestimation of the actual forces in the truss diagonals.

2.4 Conclusion

The behaviour of sandwich panels is believed to be similar to the behaviour of coupled shear walls. To predict the behaviour of partially composite sandwich panels requires knowledge of the response of the shear connectors. The tests available in the literature are for proprietary truss-type connectors. There is a need for information on commonly used generic shear connectors. Also, bond between the concrete and the insulation can transfer significant shear between the wythes. While bond is present initially, the bond may break down over time. It was concluded that:

- Tests should be performed on generic shear connectors.
- Tests should include a bond-breaker.
- Tests should avoid concrete zones and concrete in the gaps between insulation sheets.
- Full-scale wall panels should be tested, along with unit shear tests that characterize the mechanical properties of the shear connectors.
- There is no agreement on how the degree of composite action should be defined.

3 Preliminary Experimental Program

3.1 Overview of Preliminary Experimental Program

The purpose of the preliminary experimental program was to provide a better understanding of the flexural behaviour of precast / prestressed concrete sandwich panels so that definitive tests could be designed and performed. This chapter includes the testing procedure, description of test specimens, and the results for the preliminary experimental program. The testing program involved flexural tests that were performed on specimens saw-cut from three existing run-of-production sandwich panels provided by the Precast Division of Lafarge Canada Inc. Specimens A1 and A2 used expanded metal for the shear connectors while specimen A3 used M-ties. In addition to the flexural tests, the following ancillary tests were also performed; unit shear tests, coefficient of friction tests, and concrete compression strength tests.

The test results from the preliminary experimental program were used to develop a rational analytical model for concrete sandwich panels. The analytical models and the analysis results for the preliminary flexural tests are presented in Chapter 4.

3.2 Flexural Tests

Flexural tests were performed on three test specimens obtained from existing wall panels to gain a better understanding of their flexural behaviour prior to design of full-scale test specimens. The three test specimens are shown in Figure 3.1 to Figure 3.3. Test specimen A1 and A2 were saw-cut from 10 m tall panels to a length that was possible to test in the structures laboratory at the University of Alberta. The specimens were cut in such a fashion that the shear connectors were symmetric about mid-span and more importantly so that a partially composite failure would occur. Partially composite behaviour was desired so failure would occur in the shear connectors. Test specimen A3 was already a testable length and did not require saw-cutting. Because test specimen A3 was not saw-cut from a larger panel all the connection details and lifting details were still present and left intact during testing.

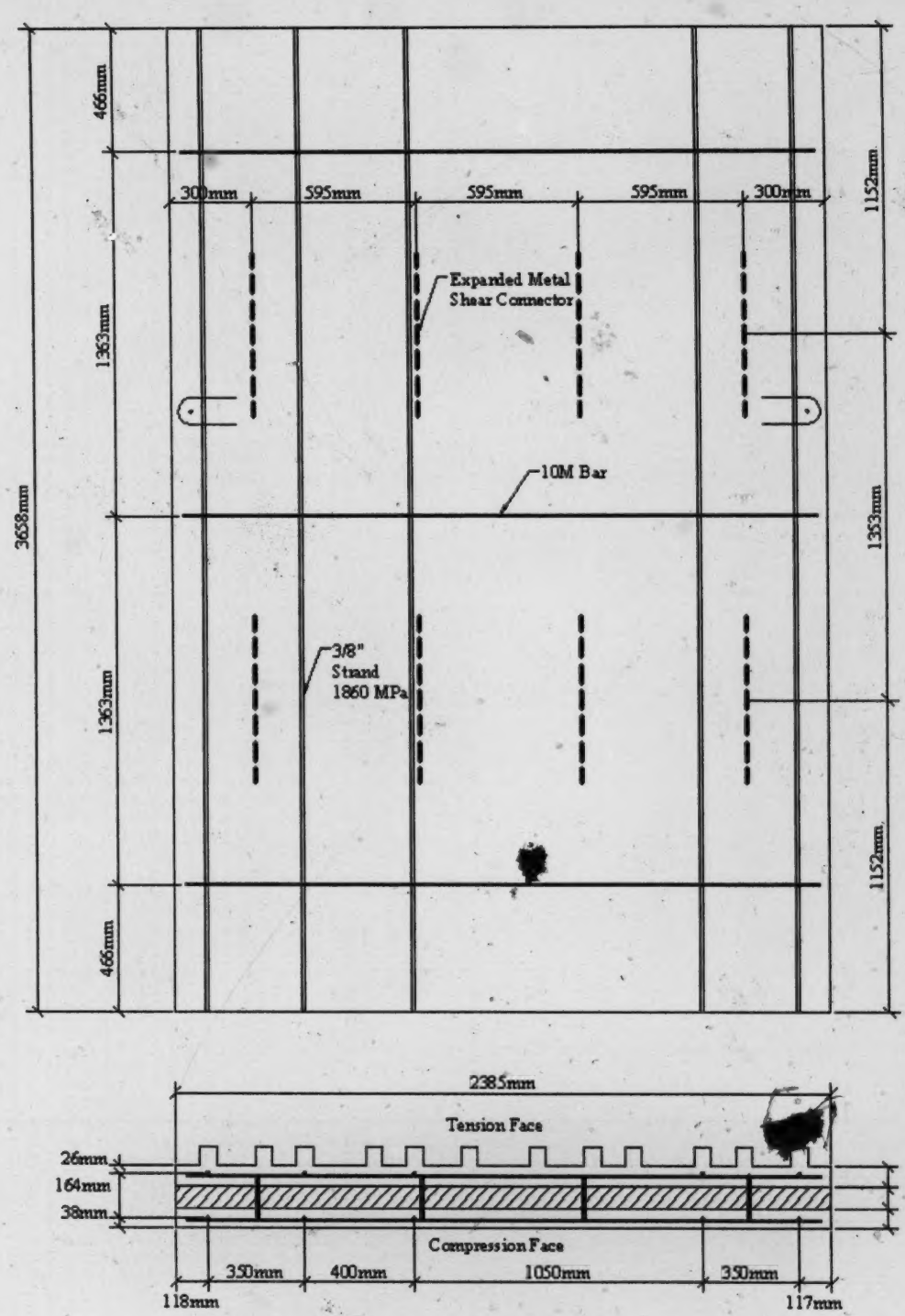

Figure 3.1 Preliminary Flexural Test Specimen A1

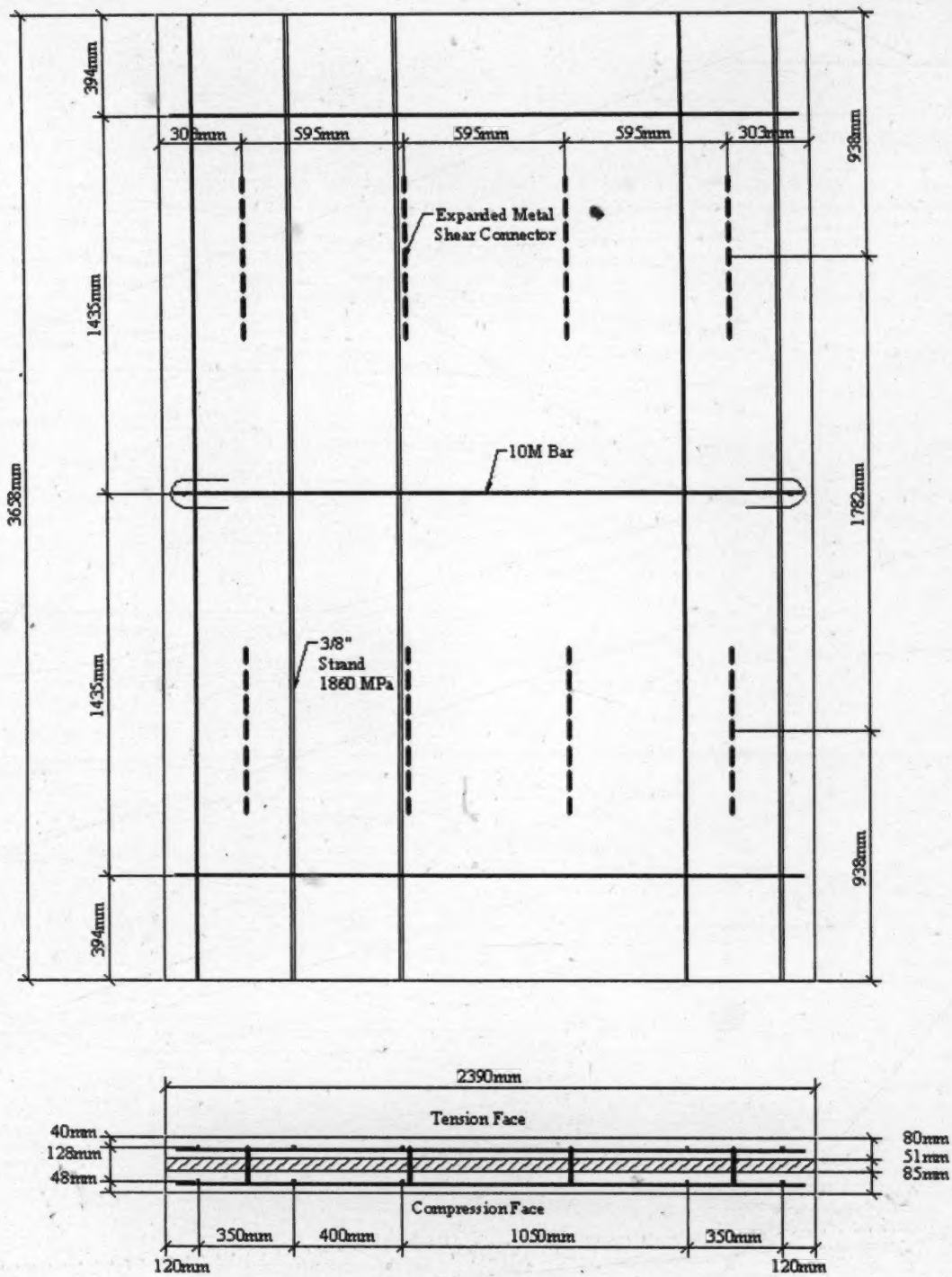

Figure 3.2 Preliminary Flexural Test Specimen A2

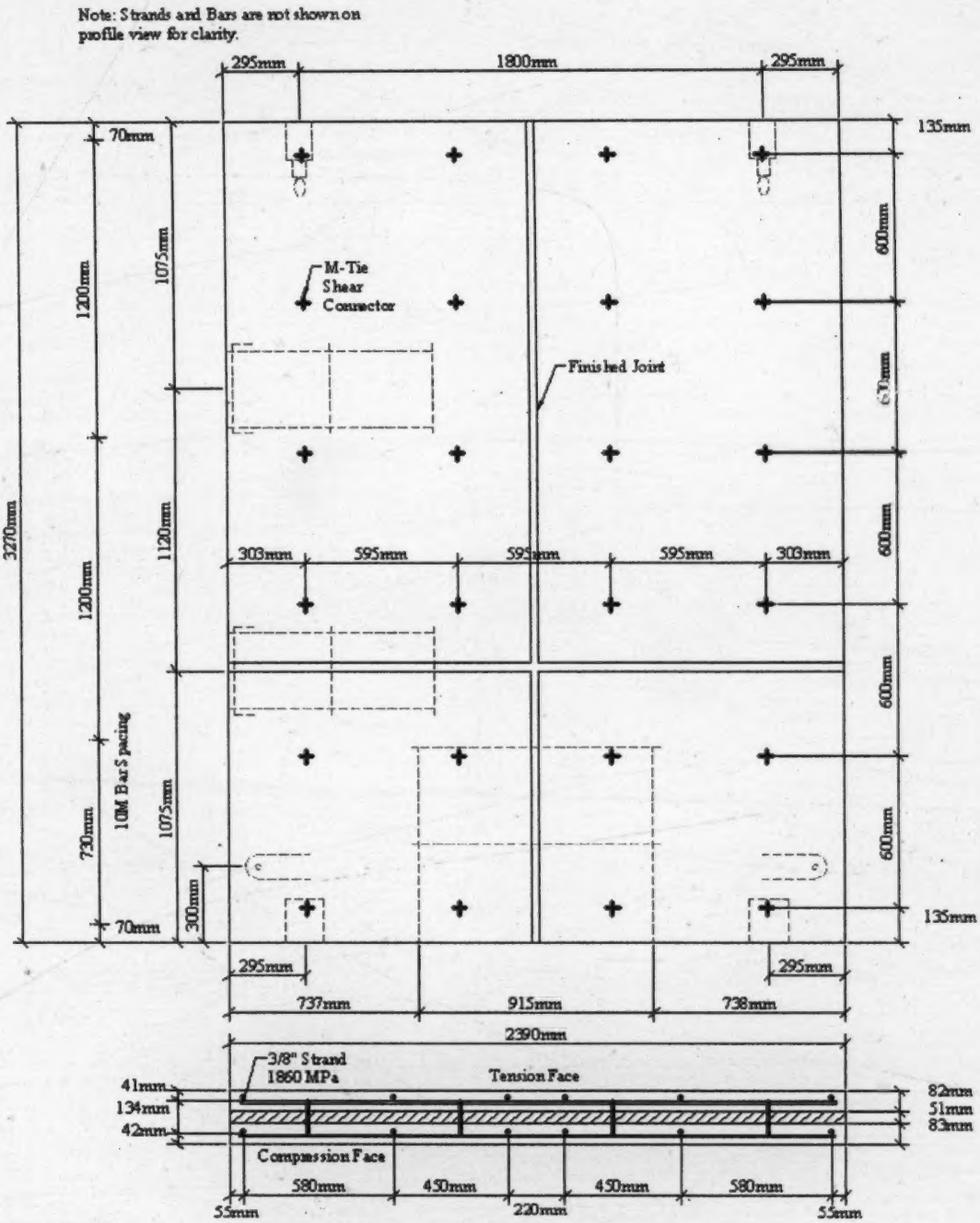

Figure 3.3 Preliminary Flexural Test Specimen A3

The sandwich panel material properties are summarized in Table 3.1. The concrete properties were measured and the nominal values were assumed for the prestressing strand properties. The detailed procedure found in the CPCI Design Manual (CPCI, 1996) was used to estimate the stress in the strands after all long term losses were accounted for. Test panels A1 and A2 used galvanized steel expanded metal shear connectors (Figure 3.4). Test panel A3 used M-tie connectors (Figure 3.5). Specimen A1 and A2 used "Deckmate" insulation. Deckmate insulation is the equivalent of the widely used Styrofoam brand insulation produced by DOW Chemical Canada Inc. Test specimen A3 used polyisocyanurate insulation with a perforated black glass reinforced mat facing on both sides.

Table 3.1 Preliminary Sandwich Panel Material Properties

	Test Specimen A1	**Test Specimen A2**	**Test Specimen A3**
Compression Wythe			
Concrete Properties			
28-day f'_c	54.8 MPa	57.0 MPa	37.1 MPa
Strand Properties			
# of Strands	5	5	6
Strand Diameter	3/8"	3/8"	3/8"
f_{pu}	1860 MPa	1860 MPa	1860 MPa
Stressed to before losses	0.60 f_{pu}	0.70 f_{pu}	0.70 f_{pu}
Stressed to after losses	0.49 f_{pu}	0.58 f_{pu}	0.58 f_{pu}
Tension Wythe			
Concrete Properties			
28-day f'_c	48.3 MPa	45.2 MPa	43.1 MPa
Strand Properties			
# of Strands	5	5	6
Strand Diameter	3/8"	3/8"	3/8"
f_{pu}	1860 MPa	1860 MPa	1860 MPa
Stressed to before losses	0.70 f_{pu}	0.70 f_{pu}	0.70 f_{pu}
Stressed to after losses	0.58 f_{pu}	0.58 f_{pu}	0.58 f_{pu}
Connector Properties			
Type	Expanded Metal	Expanded Metal	M-Tie
# of Connectors	8	8	24
Insulation Properties			
Type	Deckmate	Deckmate	Polyisocyanurate
Thickness	3"	2"	2"

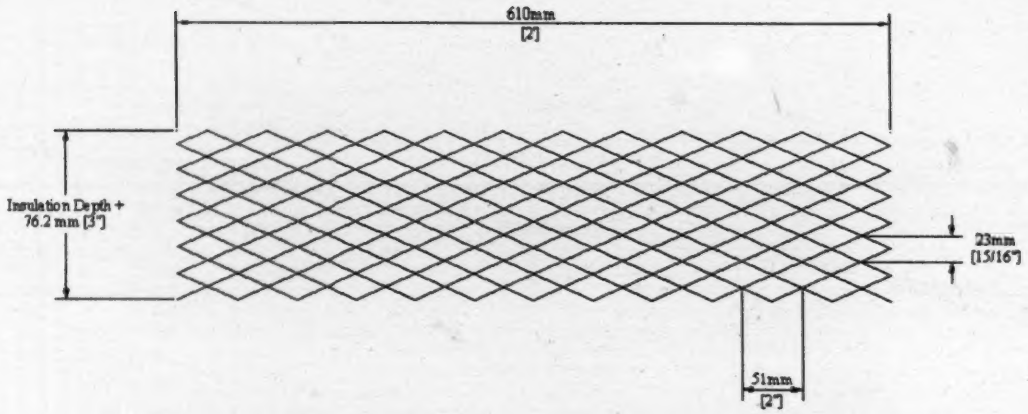

Figure 3.4 3/4"-#13 Galvanized Steel Standard Expanded Metal Shear Connector

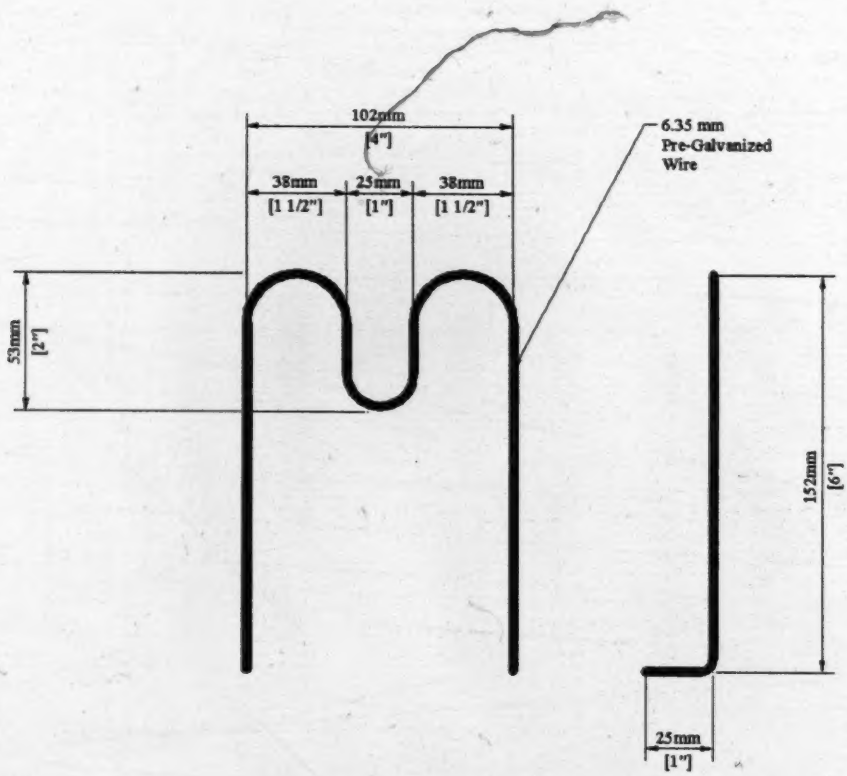

Figure 3.5 M-Tie Shear Connector

The flexural test specimens were tested in a horizontal flat position. Water bags were used to apply a uniformly distributed upward load (Figure 3.6). The preliminary flexural test set-up is shown in Figure 3.7 to Figure 3.9. The water bags were placed on the strong floor and pushed up on the specimen (Figure 3.7). The water bags were confined on the sides by the large channels and at the ends with 189 mm x 189 mm timbers that provided temporary specimen support. The end reaction members were double channels held down to the strong floor using 1/2" (13 mm) prestressing strands. The prestressing strands allowed for end rotations and end movements, therefore, providing in a simply supported boundary condition.

During the flexural tests, electronic measurements of the applied load and reactions, mid-span deflection, and end slip between the concrete wythes were taken. Manual readings were also taken to validate the electronic results. To measure the applied load both a pressure gauge and a standpipe were placed in series with the water bags. The pressure gauge, which has a maximum capacity of 69 kPa (10 psi), was calibrated using the standpipe. To measure the end reactions load cells were placed on all four prestressing strands. The mid-span deflection was measured relative to the supports. This was accomplished using an LVDT to measure the difference between an HSS section resting on top of the specimen at mid-span and an HSS section suspended between the two support reactions. Dial gauge readings were also taken, at mid-span and at each end of the specimen. To measure the end slip an LVDT measured the relative slip between the two wythes.

All three specimens were loaded in approximately 0.5 kPa increments while the specimens remained in the linear elastic range. Once the specimens began to soften the specimens were loaded in displacement increments of approximately 2 mm. Loading continued until the specimen reached failure and the load carrying capacity dropped off. During unloading periodic readings were taken to obtain the unloading curve. The flexural test results are reported along with the ancillary test results in section 3.4.

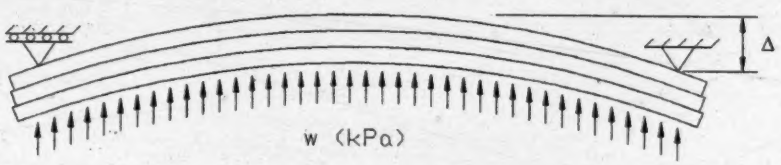

Figure 3.6 Flexural Test Schematic

Figure 3.7 Preliminary Flexural Test Set-up (Water-bags)

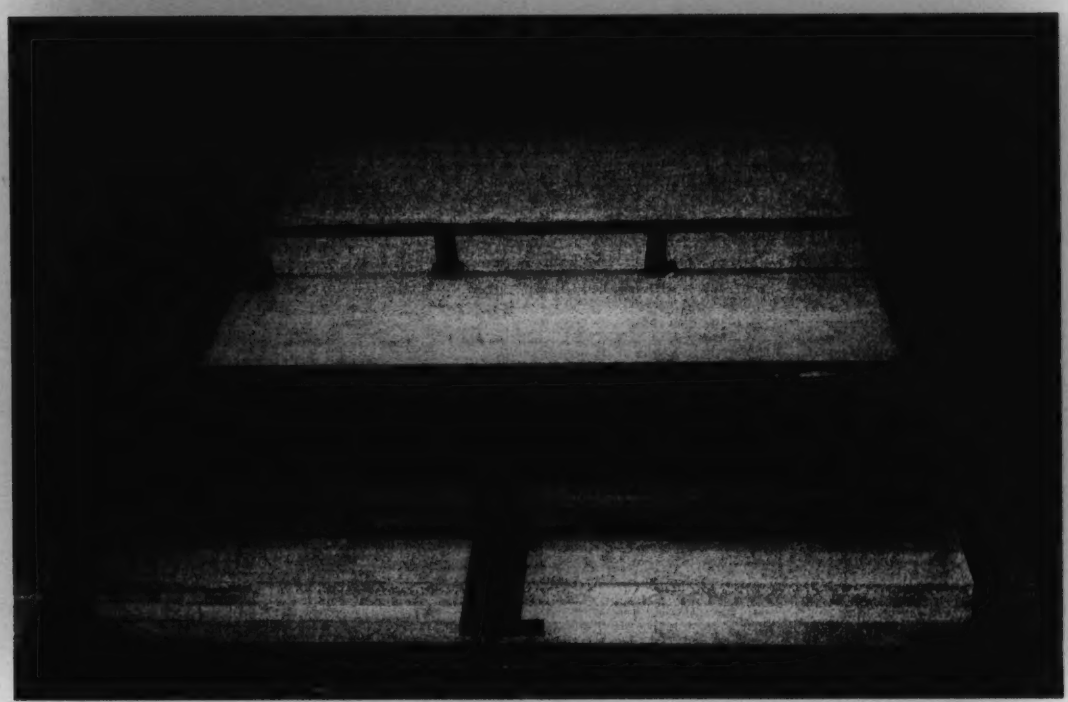

Figure 3.8 Preliminary Flexural Test Set-up (Picture)

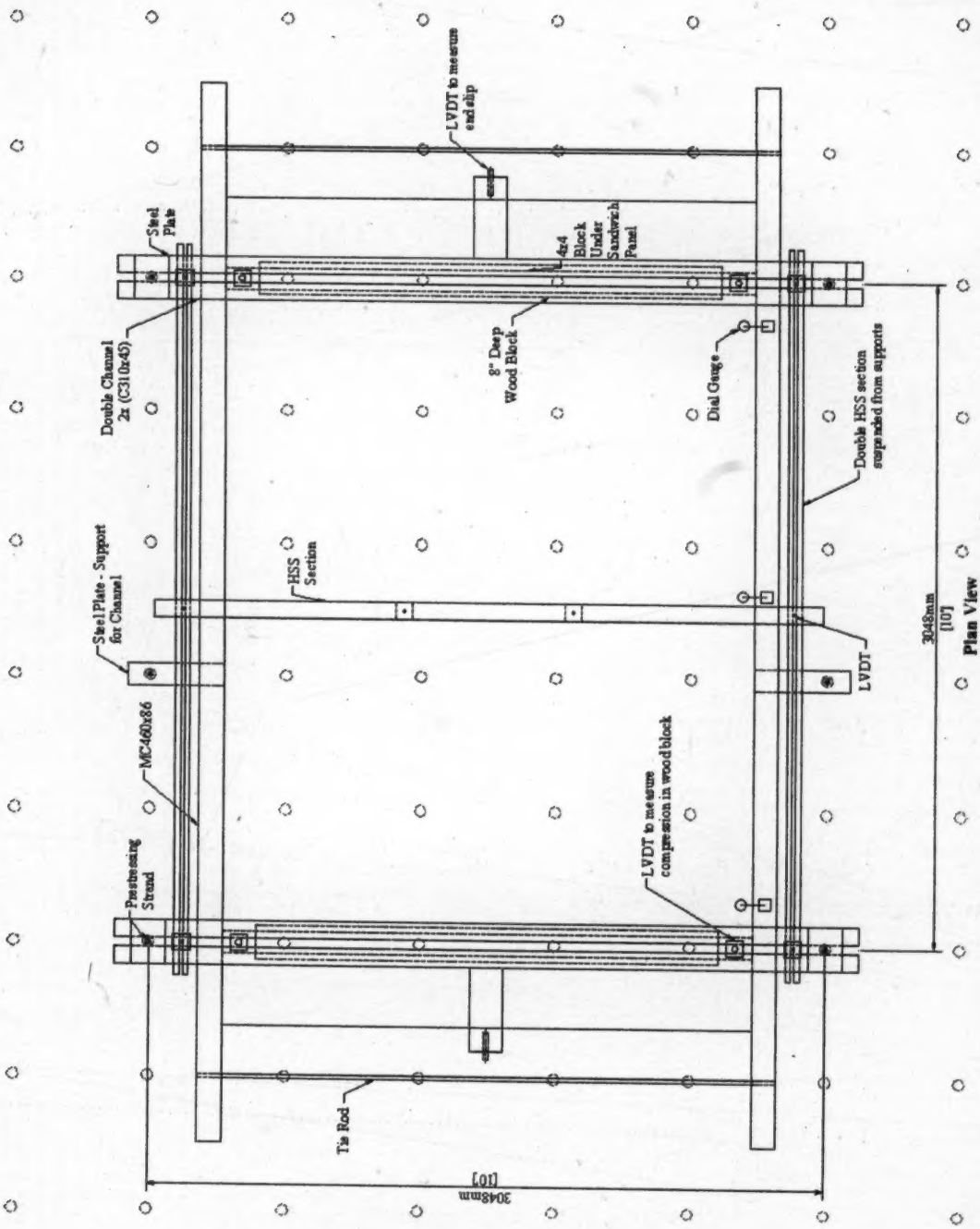

Figure 3.9 Preliminary Flexural Test Set-up (Drawing)

3.3 Ancillary Tests

This section describes the test procedures and test specimens for the ancillary tests performed to characterize the properties of the individual sandwich panel components. The ancillary tests performed included; unit shear tests, coefficient of friction tests, and concrete compression tests.

3.3.1 Unit Shear Tests

To be able to predict the flexural capacity of sandwich panels it is necessary to know the characteristics of the individual components. The unit shear test characterizes the shear force vs. shear deformation behaviour of the shear connectors. Unit shear tests were performed on specimens saw-cut from the same wall panels as flexural specimens A1 and A2 to obtain information on the expanded metal shear connectors. The specimens were approximately 380 mm wide and 1050 mm in length and contained one shear connector. As a result of saw-cutting the bond between the insulation and the concrete was compromised.

Shear tests were previously performed on specimens containing M-Tie shear connectors (Richmond, 1997). The specimens contained a polyethylene bond-breaker between the insulation and the concrete. These results were used to characterize the behaviour of the shear connectors used in flexural test specimen A3.

To perform the unit shear test the shear connectors are put into direct shear by pushing the top wythe of concrete relative to the bottom wythe of concrete (Figure 3.10). The unit shear test set-up is shown in Figure 3.11 and Figure 3.12. The bottom wythe of concrete is restrained from moving and the top wythe of concrete is pushed using a jack. Both the jack and the bottom restraint reacted against steel columns that were tied down to the strong floor. The specimens experienced an overturning moment that was resisted by tying down the specimen close to the jacking force. Load cells were used to measure both the jacking force as well as the hold down force. It was necessary to measure the hold down force in order to correctly determine the shear force experienced by the shear connectors. A frictional force exists between the insulation and the concrete due to the hold down force and weight of the upper wythe. To account for this frictional force the relation $V = \mu N$ is used, where μ is determined from the coefficient of friction tests

described in section 3.3.2 and N is the measured hold down force plus the weight of the upper wythe. Two dial gauges were also used to measure Δ, the relative displacement between the top and bottom concrete wythes. The average displacement was determined. Using the test data, a load per shear connector excluding friction vs. displacement curve can be developed to characterize the behaviour of the shear connectors. The unit shear test results are reported with the results of the corresponding flexural tests in section 3.4.

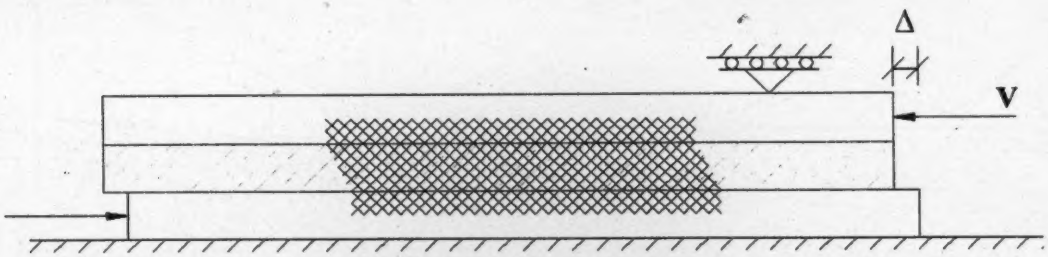

Figure 3.10 Unit Shear Test Schematic

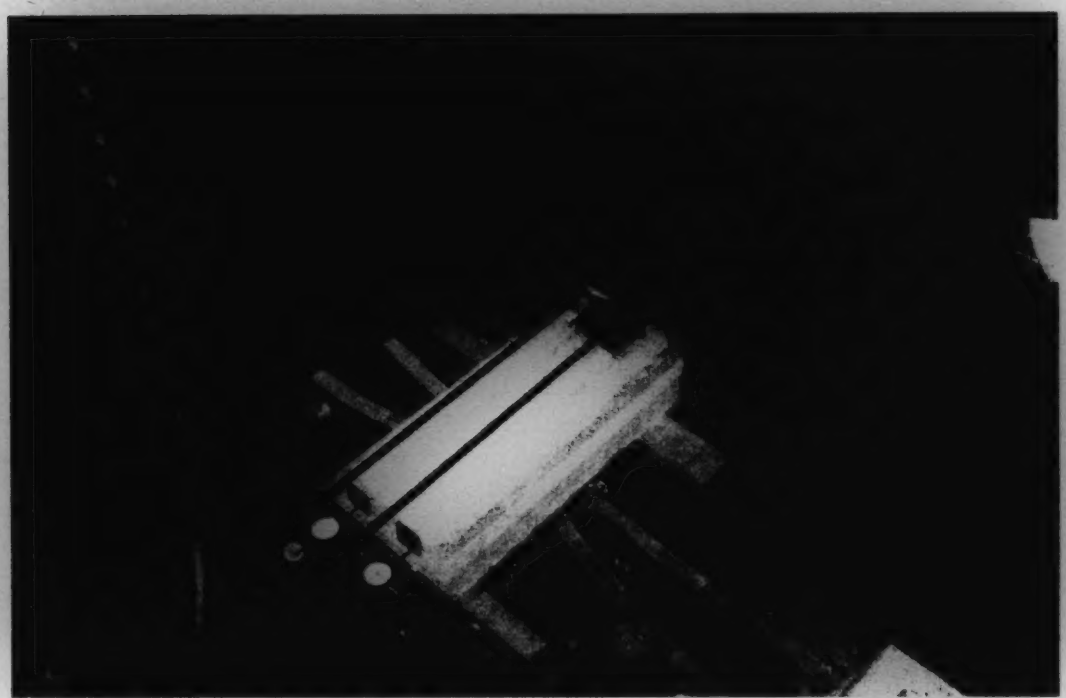

Figure 3.11 Preliminary Unit Shear Test Set-up (Picture)

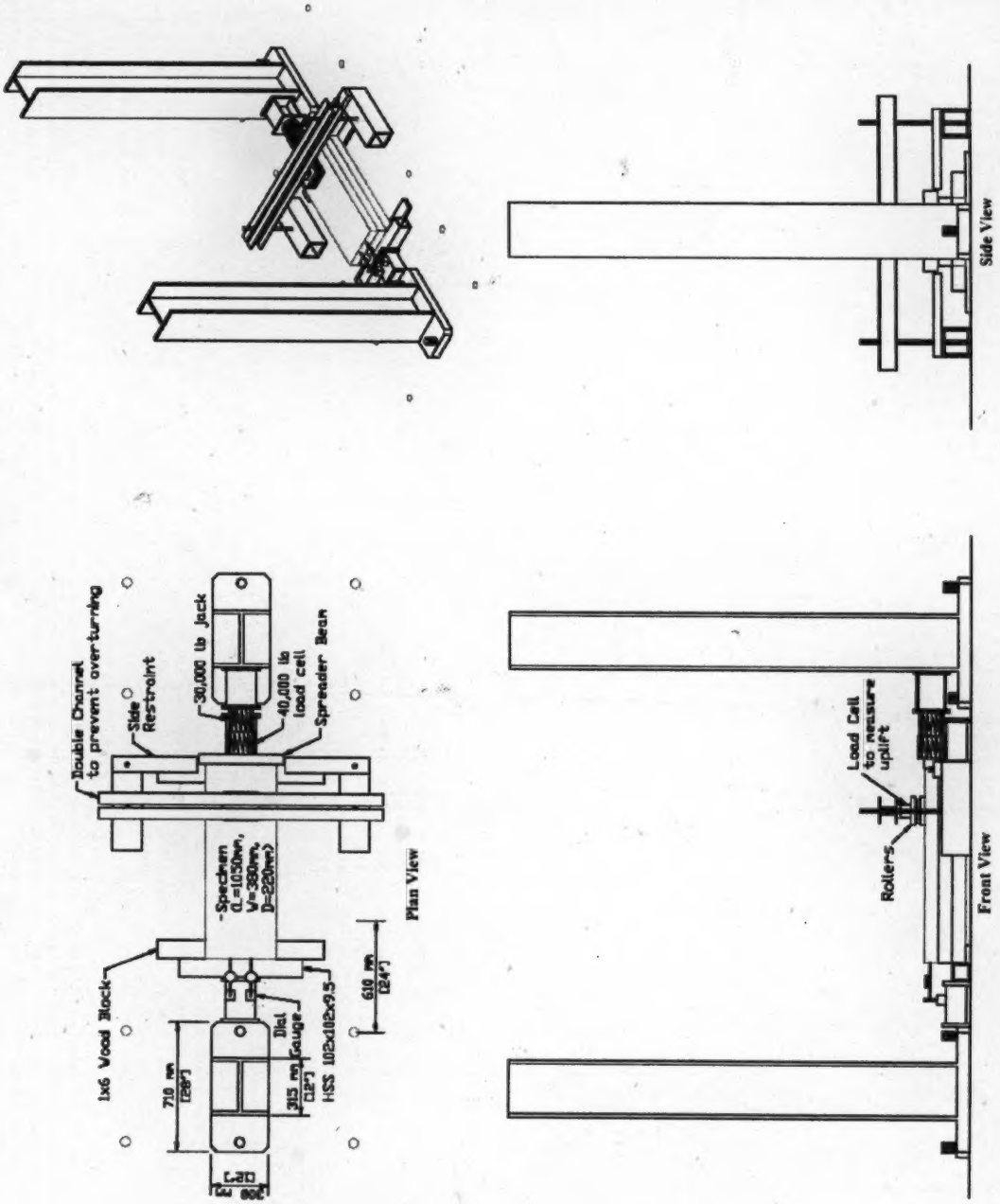

Figure 3.12 Preliminary Unit Shear Test Set-up (Drawing)

3.3.2 Coefficient of Friction Tests

During production of the sandwich panels, the concrete bonds to the insulation. As a result, a significant amount of shear can be transferred through the insulation creating a highly composite wall panel. With time it is possible that the bond between the concrete and insulation weakens. For this reason, designers usually ignore the contribution of the insulation to composite action during in-service and ultimate loading. After the bond fails, shear can still be transferred through the insulation because a frictional force exists between the insulation and the concrete. It was desired to measure the coefficient of friction between the concrete and the insulation so that this frictional force could be determined. The frictional force can be calculated using the following equation:

$$V_{insul} = \mu \times N \qquad [3.1]$$

Where: V_{insul} = frictional force (kN)
μ = coefficient of friction
N = normal force (kPa x Area = kN)

Coefficient of friction specimens containing no shear connectors were cut from the same wall panels as flexural specimens A1 and A2. The specimens were 380 mm wide and 1050 mm long. In the case of flexural test specimen A3 a separate test specimen was cast using the same style of insulation and concrete mix as the flexural test specimen. This was necessary because flexural test specimen A3 was already a testable length and therefore no left over panel was available to obtain coefficient of friction test specimens from. Prior to testing, the bond between the insulation and concrete was broken.

To determine the coefficient of friction one end of the specimen was slowly raised until slip occurred between the insulation and the concrete. At this time, the inclination angle of the specimen was measured with respect to the horizontal. The coefficient can then be calculated using the following equation:

$$\mu = \tan\theta \qquad [3.2]$$

Where: μ = coefficient of friction
θ = inclination angle at initial slip

Figure 3.13 shows the interface between the insulation and the concrete. It can be seen from this picture that air pockets develop between the bottom wythe of concrete and the insulation. These air pockets develop because the insulation is placed on the bottom wythe of concrete trapping air between the insulation and the concrete. As a result, the bond between the insulation and bottom wythe of concrete is usually weaker than the bond between the insulation and the top wythe of concrete. Therefore, the coefficient of friction between the insulation and bottom wythe of concrete was measured.

Figure 3.13 Concrete to Insulation Interface with Air Pocket

The results of the coefficient of friction test are reported with the results of the corresponding flexural tests in section 3.4.

3.3.3 Concrete Compression Tests

The compressive strength of the concrete used in the flexural test specimens was obtained from the quality control records. The concrete cylinders had a 4" diameter and were moist cured for 28 days prior to testing. No cylinders were available to determine the concrete strength at the time of the panel tests. The concrete strengths are reported in Table 3.1.

3.4 Test Results & Observations

This section presents the results for series A1, A2 and A3. Each test series consists of the flexural test and associated ancillary tests. The unit shear tests are presented as a shear load per connector vs. shear displacement curve. The flexural tests are presented as a transverse load vs. mid-span deflection curve and as an end slip vs. mid-span deflection curve. Manual and electronic readings were taken during the preliminary flexural tests. The two sets of readings agreed within 2%, so for clarity only the electronic readings are presented. Load cells were used to measure the end reactions. The pressure calculated from the load cells agreed within 4% of the pressure calculated from the pressure gauge. The average pressure was used. The panels were tested spanning north – south, so the deflection vs. end slip graph has separate curves showing the end slip at the north end and south end of the panels. The drawings of the test specimens all have the north end of the specimen at the top of the page. Pictures of the crack patterns formed on the tension face of the wall panels are also provided.

3.4.1 Test Series A1

Figure 3.14 shows the net load vs. mid-span deflection curve for the flexural test. The net load being the upward fluid pressure acting on the wall, minus the downward acting self-weight of the wall. The specimen exhibited ductile behaviour. The specimen remained linearly elastic up to a pressure of 20.3 kPa and a deflection of 6.19 mm. The specimen reached an ultimate capacity of approximately 24.9 kPa and maintained this load until a mid-span deflection of 33.2 mm was achieved. The load carrying capacity of the panel then dropped to a value near 19.8 kPa. This value was maintained until the test was finally stopped at a deflection of 63.2 mm. Figure 3.15 shows the end slip vs. mid-span deflection. It is clear from this graph that the failure occurred in the shear connectors on the south half of the specimen. The end slip on either side of the panel remained virtually the same up to a value of 1.71 mm. At this end-slip, failure of the connectors in the south half of the panel started. The end slip was 10.15 mm at the south end of the panel when the test was finally stopped.

Figure 3.14 Flexural Test - Load vs. Mid-span Deflection (Test Series A1)

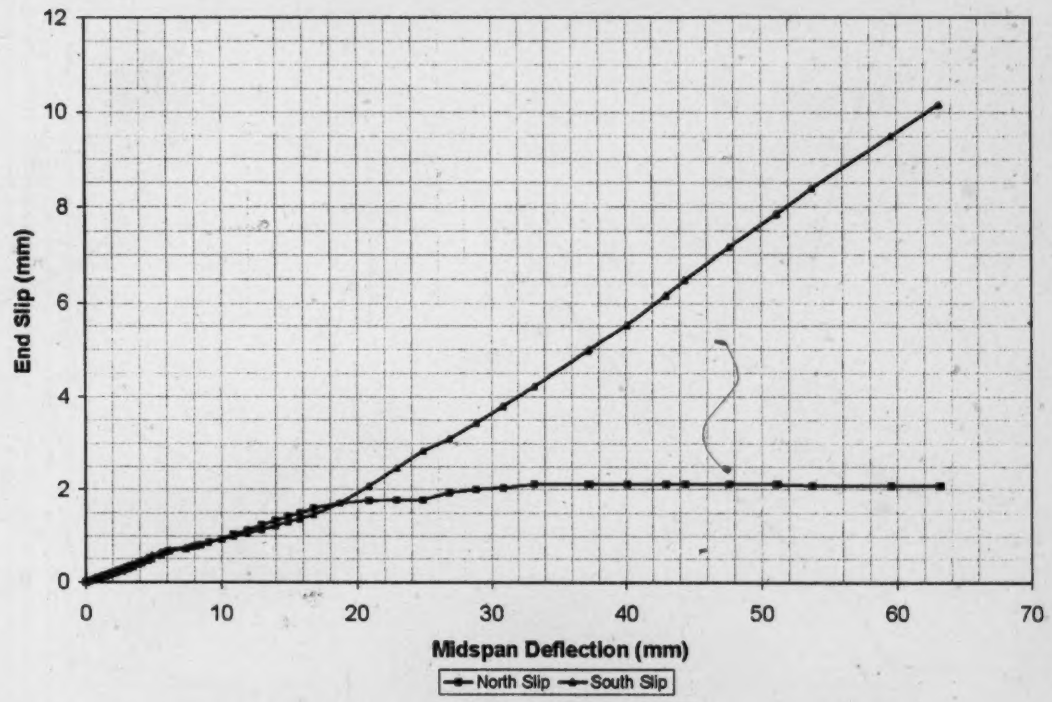

Figure 3.15 Flexural Test - End Slip vs. Mid-span Deflection (Test Series A1)

Figure 3.16 shows the crack pattern for flexural test specimen A1. The primary flexural cracks identified as crack 1 in the figure were first observed at a mid-span deflection of 6.2 mm. This corresponds to the drop in the load vs. mid-span deflection curve immediately after the linearly elastic region. Crack 2 occurred at a mid-span deflection of 47.6 mm. This is where the ultimate load carrying capacity drops off.

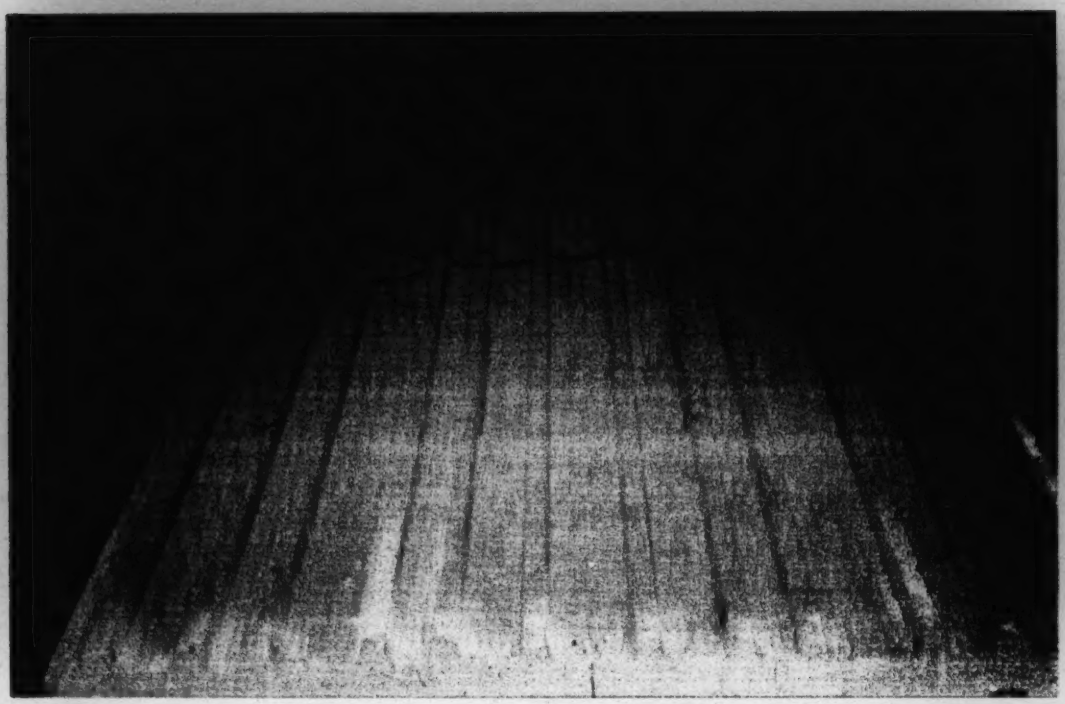

Figure 3.16 Crack Pattern (Test Series A1)

Figure 3.17 shows the load vs. displacement curves for the unit shear tests. Two specimens were saw-cut for the purposes of unit shear tests. From the graph it can be seen that there was a difference in behaviour of the two specimens. The ultimate capacity of test specimen A1-1 was 23.1 kN and this occurred at an average displacement of 1.88 mm. The ultimate capacity of test specimen A1-2 was 28.0 kN and this occurred at an average displacement of 2.35 mm. When performing the analysis the average of the two curves was used.

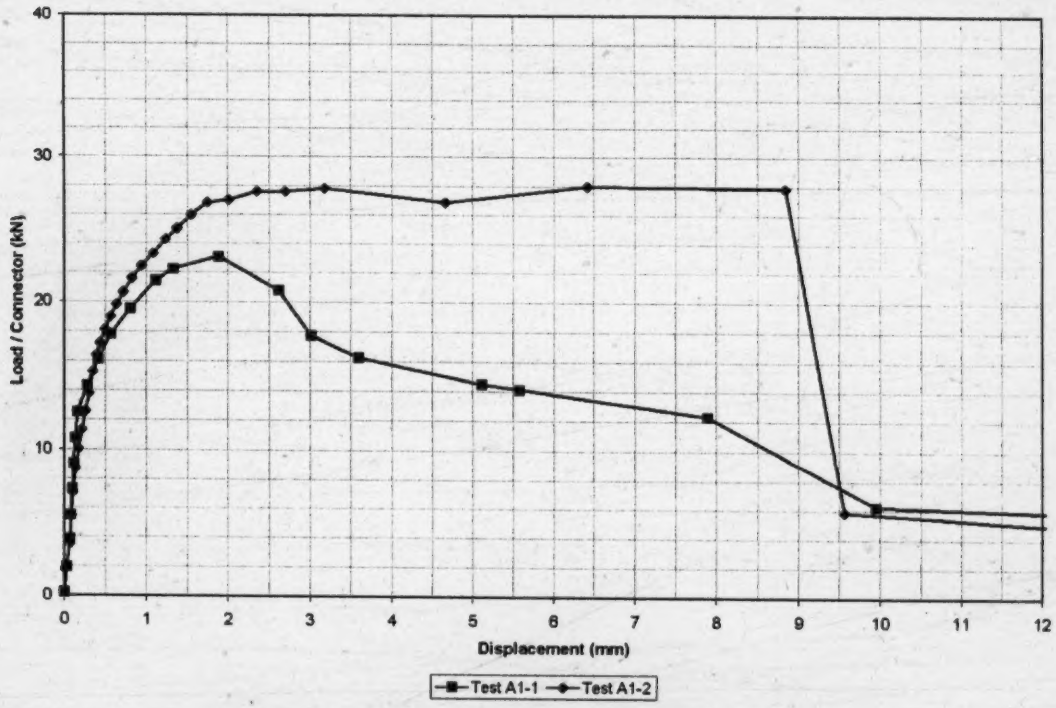

Figure 3.17 Unit Shear Test - Load vs. Displacement (Test Series A1)

Two specimens were used for the coefficient of friction tests. Slip occurred at an average angle of 33° to the horizontal. As a result, the coefficient of friction between the insulation and the concrete was calculated to be 0.65.

3.4.2 Test Series A2

The results from the flexural test are illustrated in Figure 3.18 and Figure 3.19. Figure 3.18 shows the net load vs. mid-span deflection curve. The test specimen exhibited ductile behaviour. The specimen remained linearly elastic up to a load of 17.0 kPa with a corresponding mid-span deflection of 7.01 mm. The load increased to 18.9 kPa before the specimen began to deflect drastically. The load increased gradually to a value of 22.8 kPa with a corresponding mid-span deflection of 56.02 mm. At this point the test specimen began to loose stiffness and the test was stopped. The end slip vs. mid-span deflection is shown in Figure 3.19. The connectors at both ends of the panel underwent similar deformations during loading. Failure finally occurred in the shear connectors at the north end of the panel. The shear connectors on the north half of the panel underwent 6.68 mm of deformation prior to stopping the test.

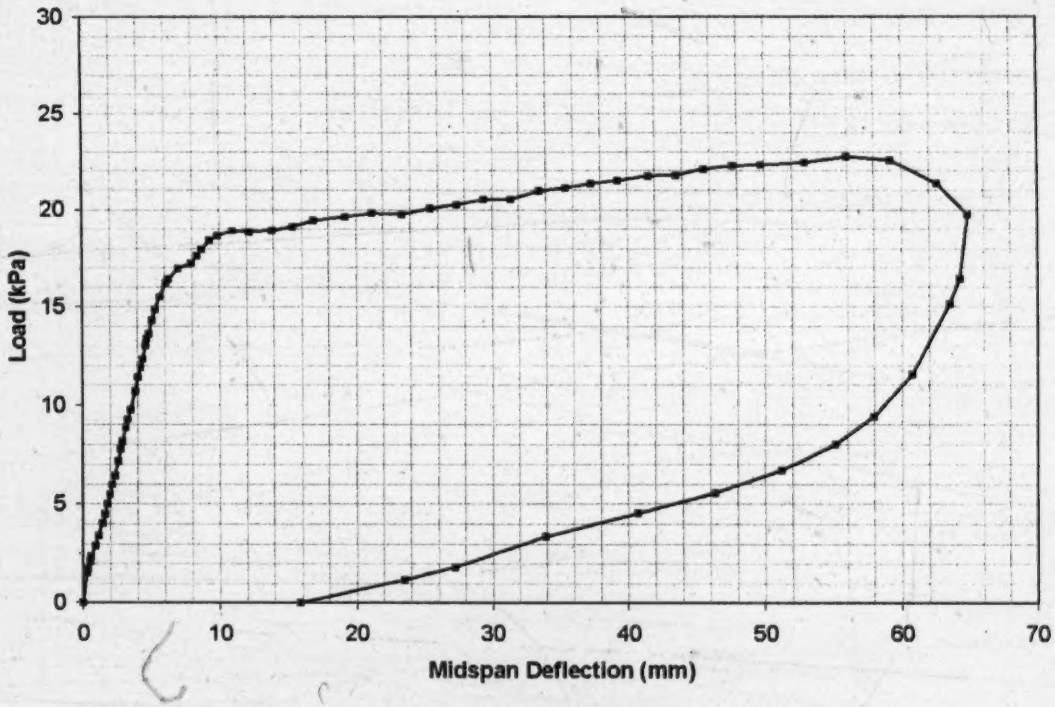

Figure 3.18 Flexural Test - Load vs. Mid-span Deflection (Test Series A2)

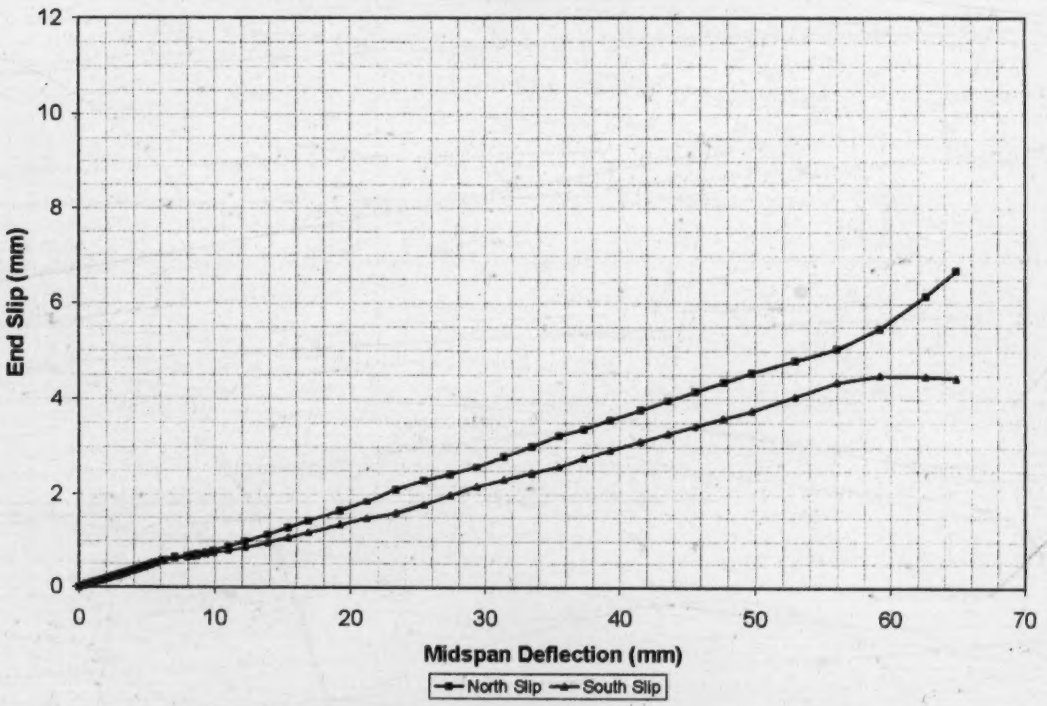

Figure 3.19 Flexural Test - End Slip vs. Mid-span Deflection (Test Series A2)

The crack pattern that developed in flexural test specimen A2 is shown in Figure 3.20. Crack 1 was first observed when the mid-span deflection was 7.01 mm. A reduction in stiffness can be seen on the load vs. mid-span deflection curve when this first crack occurred. Crack 2 began to form at a mid-span deflection of 8.35 mm. This crack eventually opened up and became the primary flexural crack. At the end of the test this crack was about 5 mm wide. The cracks identified as 3 occurred when the panel had a deflection of 39.28 mm. At a mid-span deflection of 59.20 mm crack marked 4 formed at the north end of the specimen.

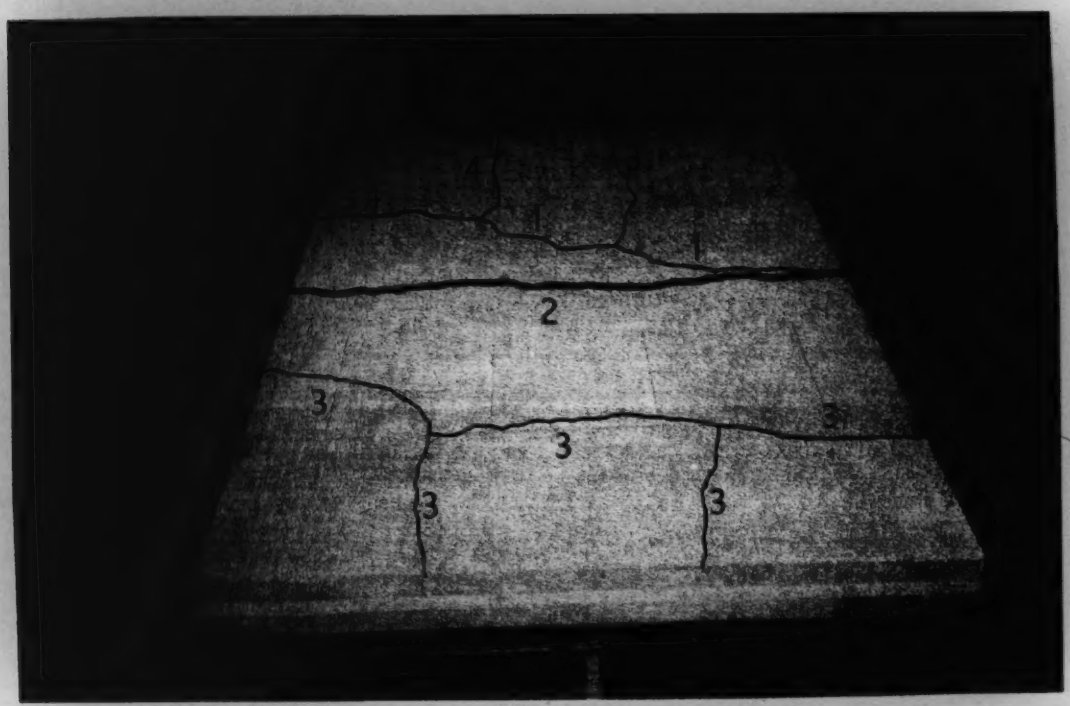

Figure 3.20 Crack Pattern (Test Series A2)

The load vs. displacement curves for the unit shear tests are shown in Figure 3.21. Three unit shear test specimens were saw-cut and tested. The results from the shear tests agreed more closely than those of test series A1. The ultimate capacity of test specimen A2-1 was 39.8 kN at a displacement of 5.36 mm. The ultimate capacity of test specimen A2-2 was 36.9 kN at a displacement of 5.33 mm. Test specimen A2-3 obtained an ultimate capacity of 39.5 kN at a displacement of 5.25 mm. The average of the three curves was used for subsequent analysis.

The load vs. displacement curves for the unit shear tests had three distinct phases. The curve is linearly elastic up to an average load of 27.9 kN and an average displacement of 0.54 mm. At this time the shear connector begins to plastically deform. During the plastic deformation, the capacity of the connector continues to increase nearly linearly to an average value of 38.7 kN. At this point the connector has undergone an average deformation of 5.32 mm. The connector then fails, and maintains an average residual capacity of 9 kN.

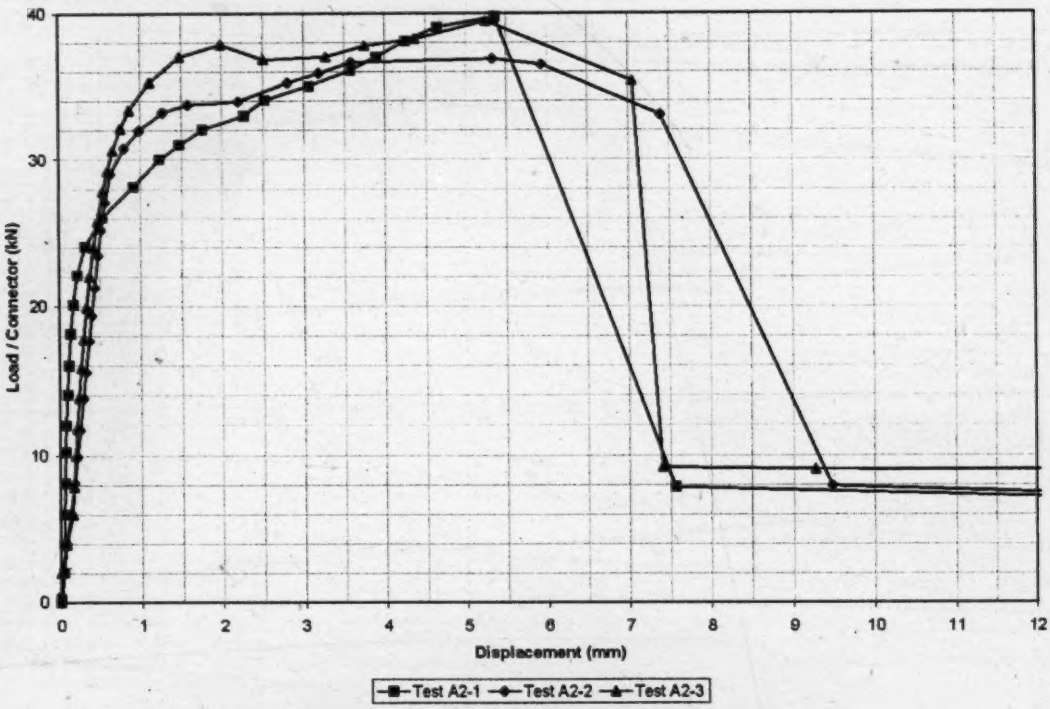

Figure 3.21 Unit Shear Test - Load vs. Displacement (Test Series A2)

Two specimens were used to measure the coefficient of friction between the insulation and the concrete. The average angle between the specimen and the horizontal plane when slip occurred was 30°. The coefficient of friction was then calculated to be 0.58.

3.4.3 Test Series A3

The net load vs. mid-span deflection curve for flexural test specimen A3 is shown in Figure 3.22. This specimen exhibited ductile behaviour. The specimen remained linearly elastic up to a net load of 12.9 kPa and mid-span deflection of 4.67 mm. The load was gradually increased to a value of 20.9 kPa with a corresponding mid-span deflection of 53.65 mm. At this time the stiffness dropped off and the test was stopped. Figure 3.23 shows the end slip vs. mid-span deflection curve for the flexural test. Most of the end slip occurred in the south half of the specimen.

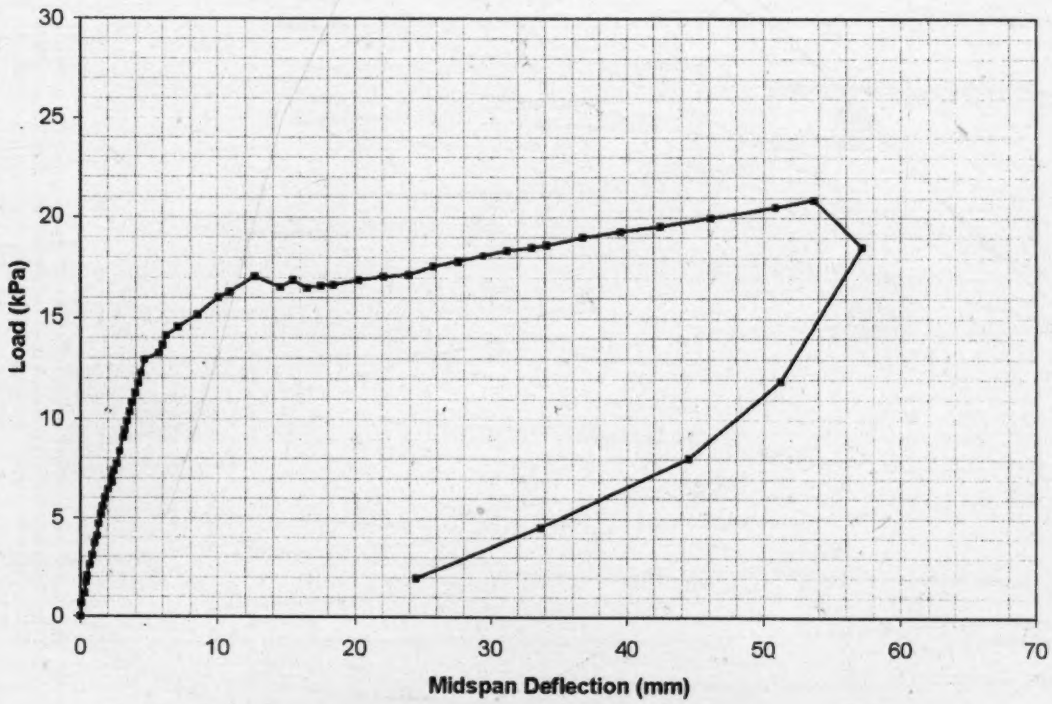

Figure 3.22 Flexural Test - Load vs. Mid-span Deflection (Test Series A3)

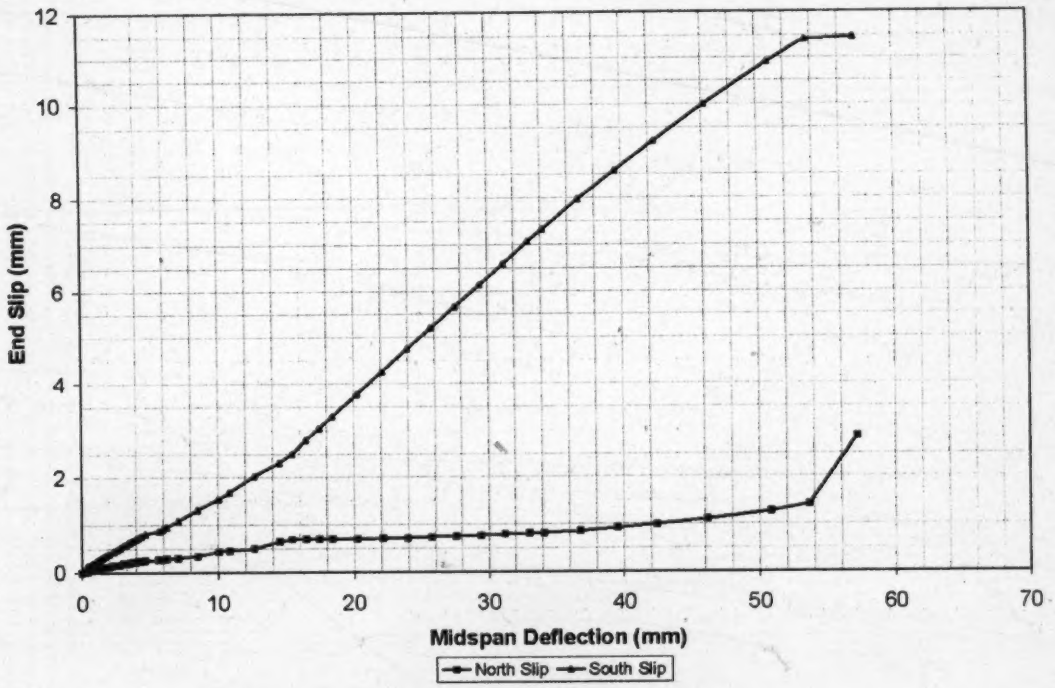

Figure 3.23 Flexural Test - End Slip vs. Mid-span Deflection (Test Series A3)

Figure 3.24 shows the flexural crack pattern that developed during test A3. The primary flexural crack (crack 1) formed at the transverse reveal located on the south half of the specimen. The crack was first observed at a mid-span deflection of 4.67 mm. The crack formation corresponds with a reduction in stiffness on the load vs. mid-span deflection curve immediately after the linearly elastic region. Cracks 2 and 3 formed at mid-span deflections of 10.86 mm and 14.53 mm respectively.

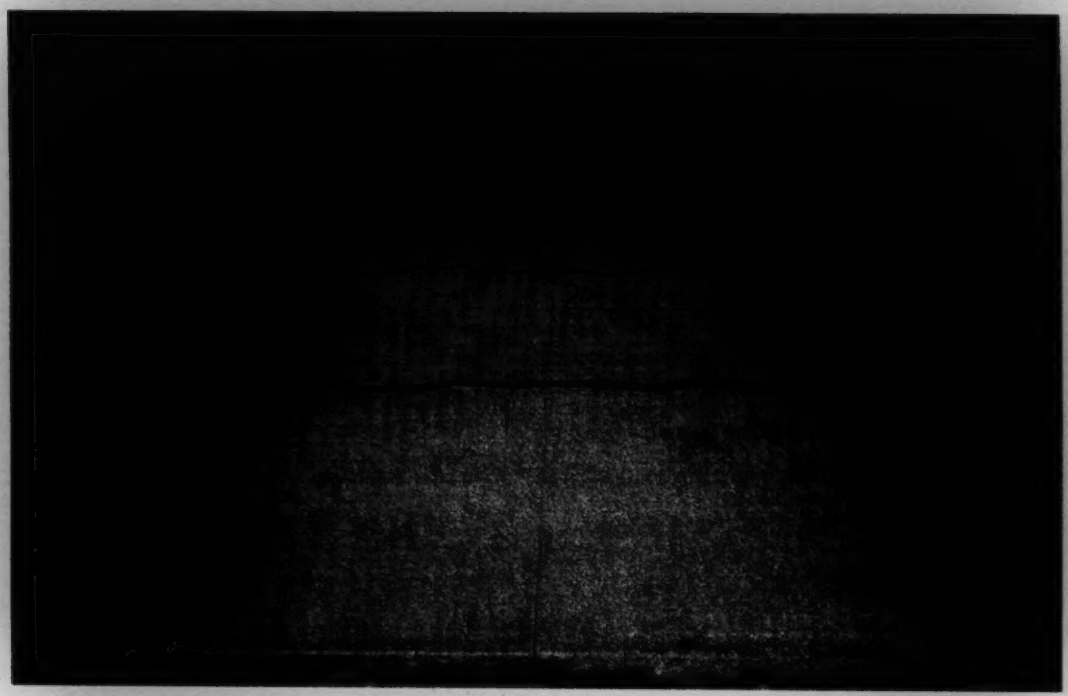

Figure 3.24 Crack Pattern (Test Series A3)

The unit shear test results shown in Figure 3.25 were from work performed at the University of Alberta (Richmond, 1997). The test specimens were produced with four M-Tie connectors and polyisocyanurate insulation. A bond breaker was used between the insulation and the concrete in order to accurately determine the characteristics of the connectors. The graph shows the load vs. displacement relationship for a single connector.

Test specimen A3-1 reached its ultimate capacity of 17.2 kN with a corresponding deformation of 25.16 mm. Specimen A3-2 had an ultimate capacity of 21.7 kN and a displacement of 25.00 mm. The last specimen reached its ultimate capacity of 22.9 kN after deforming 25.90 mm. The average of the three curves was used for subsequent analysis. It should be pointed out that these results, based on Richmond (1997), were for different lots of M-ties and different batches of concrete. Because the entire panel was used for the flexural tests, it was not possible to perform unit shear tests for the material in the panel.

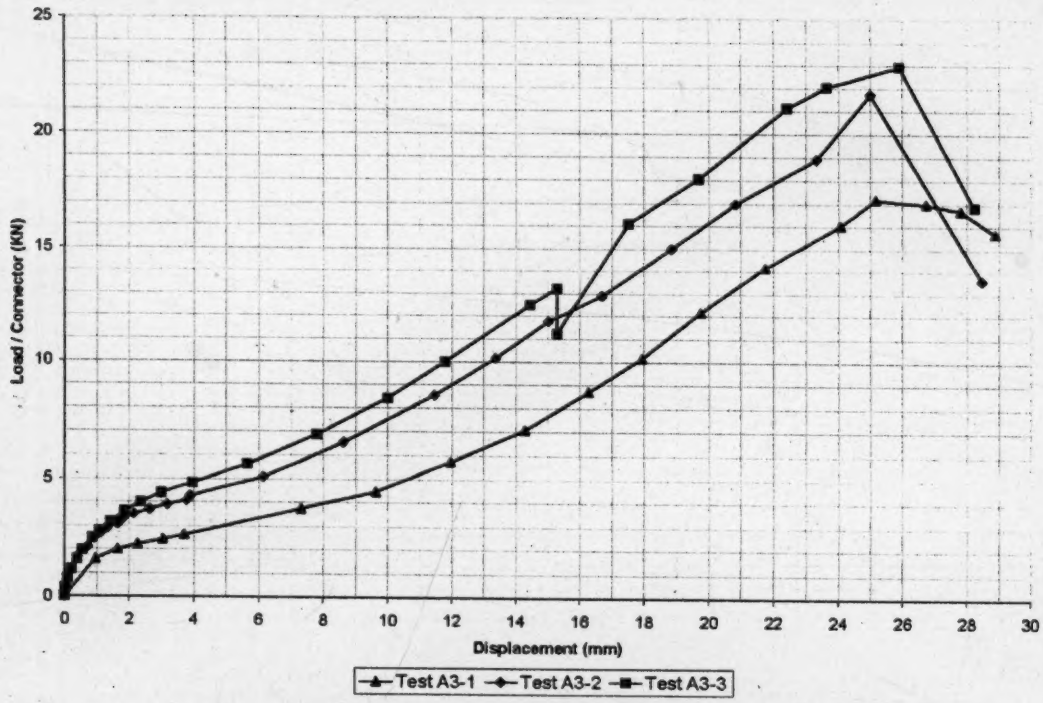

Figure 3.25 Unit Shear Test - Load vs. Displacement (Test Series A3)

To measure the coefficient of friction between the polisocyanurate insulation and concrete, test specimens were poured at a later date. The bond between the insulation and the concrete was broken prior to performing the test. Slip occurred at an average angle of 42.8°. The coefficient of friction was then determined to be 0.925.

3.4.4 General Observations

All three flexural test specimens demonstrated ductile failures. The specimens exhibited a semi-composite behaviour with failure occurring in the shear connectors. The degree of composite action will be discussed in the analysis.

From the unit shear tests it is apparent that the insulation thickness affects the capacity of the connectors. Increasing the insulation thickness reduces the capacity of the connector. Test series A1 and A2 both used the same style of expanded metal but the insulation thickness was different. Test series A1 had a 76.2 mm thick insulation and the connectors' average ultimate capacity was found to be 25.6 kN. Test series A2 had a 50.8 mm insulation thickness and the connectors' average ultimate capacity was found to

be 38.7 kN. This behaviour was also observed by Richmond (1997) for M-tie connectors.

The expanded metal connectors had a relatively large capacity with a shear displacement of approximately 2 mm. The connectors maintained their capacity through a shear deformation of approximately 6 mm. This behaviour is desirable for shear connectors to be used in semi-composite panels. The M-Tie connectors deform under relatively small loads. The connectors undergo large deformations before they carry substantial loads. The M-Tie connectors underwent a deformation of 25 mm prior to reaching their ultimate capacity. This behaviour is desirable for non-composite panels. At lower loads, little shear transfer will occur through the M-Tie connectors.

The coefficient of friction tests performed on the specimens made with Deckmate insulation produced by DOW Chemical Canada Inc. were within 10% of one another. The average coefficient of friction from the two sets of tests was 0.61. This value may be used to analyse the flexural capacity of panels containing the Deckmate insulation.

4 Evaluation of Preliminary Test Results

4.1 Development of Rational Analytical Models

This chapter describes the procedure used to analyse the overall behaviour of sandwich panels. A simplified method to estimate the ultimate capacity of sandwich panels is also presented. Definitions used to describe the degree of composite action for service load deflections and for ultimate capacity are considered. The results for the preliminary experimental program are evaluated using the methods discussed.

4.2 Overall Behaviour of Test Panels

The calculations used to estimate the overall behaviour of the test panels is shown, in detail, in Appendix A. The first step in the procedure is to develop the moment vs. curvature diagrams for the individual concrete wythes. These diagrams are used to estimate the mid-span deflection of the panels for an assumed curvature. The mid-span deflection is then used to estimate the slip between the two wythes at the end of the panels. The end slip is used to estimate the deformation of the individual shear connectors. Using the estimated deformation, the load carried by the individual shear connectors can be determined using the unit shear test results. Refining Eqn. [2.2] gives the following equation for the moment capacity of sandwich panels:

$$M_n = M_1 + M_2 + (\phi_{sc} \times V_{sc} + \phi_{insul} \times V_{insul}) \times jd \qquad [4.1]$$

but ($\phi_{sc}V_{sc} + \phi_{insul}V_{insul}$) is limited to $\phi_p A_p f_{pr}$ for tension wythe

Where: M_1 = the moment capacity of the compression wythe (including appropriate resistance factors for design)

M_2 = the moment capacity of the tension wythe (including appropriate resistance factors for design)

V_{sc} = the load carried by the shear connectors between locations of maximum moment and zero moment (i.e. half of panel)

V_{insul} = the shear transferred through the insulation between locations of maximum moment and zero moment (i.e. half of panel)

ϕ_{sc} = resistance factor for the shear connectors (1.0 for analysis and 0.65 for design)

ϕ_{insul} = resistance factor for the insulation friction (1.0 for analysis and 0.0 for design)

jd = distance between the centres of the two concrete wythes

4.3 Simplified Ultimate Capacity Analysis

Appendix B provides an example showing the simplified analysis method used to estimate the ultimate capacity of sandwich panels. The calculations were done in accordance with the CPCI Design Manual (CPCI, 1996). Eqn. [4.1] is the basis for the simplified ultimate capacity analysis.

From the overall behaviour analysis, it was observed that the shear connectors fail prior to the individual wythes reaching their ultimate moment capacity. The concrete wythes typically reach their cracking moment or slightly beyond their cracking moment prior to the shear connectors failing. As a result M_1 and M_2 are limited to the cracking moments.

4.4 Description of Composite Action

There are many different definitions used to define the degree of composite action. Eqn [2.4] can be used to calculate the degree of composite action when interested in service load deflections. To use this equation it is necessary to have performed tests on the sandwich panel of interest.

The following definition of the degree of composite action is proposed:

$$\%\ Composite = \frac{(\phi_{sc} \times V_{sc}) + (\phi_{insul} \times V_{insul})}{\phi_p A_p f_{pr(tension\ wythe)}} \qquad [4.2]$$

but not greater than 100%

Where: ϕ_{sc} = 1.0 for analysis and 0.65 for design
V_{sc} = the load carried by the shear connectors between locations of maximum moment and zero moment (i.e. half of panel)
ϕ_{insul} = 1.0 for analysis and 0.0 for design
V_{insul} = shear transferred through the insulation between locations of maximum moment and zero moment (i.e. half of panel)
ϕ_p = 1.0 for analysis and 0.9 for design
A_p = Area of prestressing steel in tension wythe
f_{pr} = stress in the prestressed reinforcement in the tension wythe when member is at ultimate resistance

This definition is philosophically consistent with that used in CSA S16.1 (CSA, 1994) and is primarily relevant to ultimate capacity calculations. The definition in Eqn. [4.2] assumes that the shear connectors between locations of maximum moment and zero moment will provide shear forces commensurate with the shear deformations of their respective locations. For the standard expanded metal shear connectors tested, except near mid-span, there is sufficient shear deformation to mobilize the plastic shear capacity of the connectors.

When evaluating the preliminary tests, three values for composite action will be computed. The first value will be calculated using Eqn. [2.4]. The next two values will be calculated using Eqn. [4.2] with and without the contribution from the shear transferred through the insulation. The analysis example in Appendix B shows sample calculations for the degree of composite action.

4.5 Evaluation of Results

The analyses of the preliminary test results are summarized in Table 4.1. The predicted loads were calculated using the simplified analysis procedure presented in Appendix B. When performing the simplified analysis on test specimen A3 the measured end slip of 11.46 mm was used in conjunction with the unit shear test results to estimate the load carried by the shear connectors along the length of the panel. In the following subsections the analyses of the specimen's overall behaviour are presented.

Table 4.1 Evaluation of Preliminary Test Results

	Test Specimen A1	Test Specimen A2	Test Specimen A3
Loads			
w_{test}	24.90 kPa	22.75 kPa	20.89 kPa
$w_{predicted\ (Overall\ Behaviour\ Anal)}$	20.52 kPa	22.70 kPa	18.00 kPa
$w_{test} / w_{predicted\ (Overall\ Behav.)}$	1.213	1.002	1.161
$w_{predicted\ (Simplified\ Analysis)}$	19.31 kPa	21.71 kPa	16.96 kPa
$w_{test} / w_{predicted\ (Simplified)}$	1.289	1.048	1.231
% Composite			
Eqn. [2.4]	6.6 %	2.0 %	4.0 %
Eqn. [4.2]			
excluding V_{insul}	20.4 %	31.0 %	10.2 %
including V_{insul}	30.6 %	41.2 %	20.9 %

4.5.1 Test Series A1

Figure 4.1 shows the predicted net load vs. mid-span deflection curve determined following the procedure presented in Appendix A along with the tested net load vs. mid-span deflection curve. The initial stiffness of the test specimen was very close to the predicted initial stiffness. However, the test resulted in higher loads than the prediction up to a mid-span deflection of 44.0 mm. At this time, the specimen maintained a load that was close to the predicted load carrying capacity. It is believed that there was significant bond between the insulation and the concrete that resulted in the additional load carrying capacity. The bond likely deteriorated at a mid-span deflection of 33.2 mm at which point the actual capacity of the test specimen began to approach the predicted value.

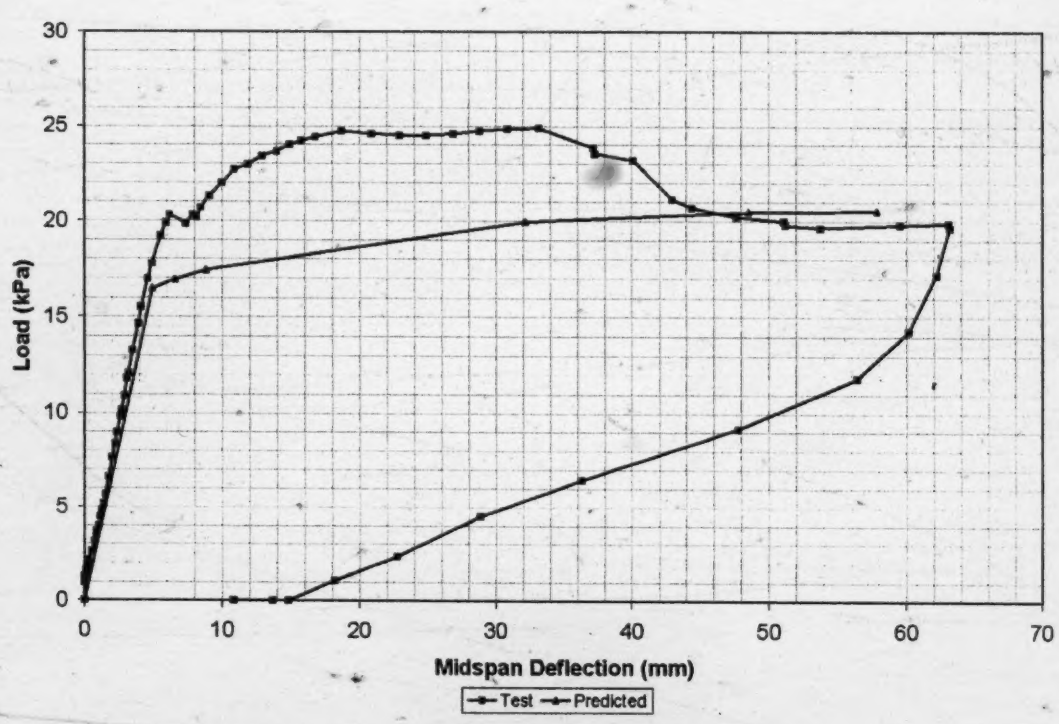

Figure 4.1 Flexural Test - Predicted Load vs. Mid-span Deflection (Test Series A1)

4.5.2 Test Series A2

The predicted net load vs. mid-span deflection curve is shown along with the tested net load vs. mid-span deflection curve for test specimen A2 in Figure 4.2. The predicted curve was developed using the procedure presented in Appendix A. The predicted curve agrees with the actual test curve. Unlike test specimen A1 it appears that the bond between the insulation and the concrete had been broken prior to testing.

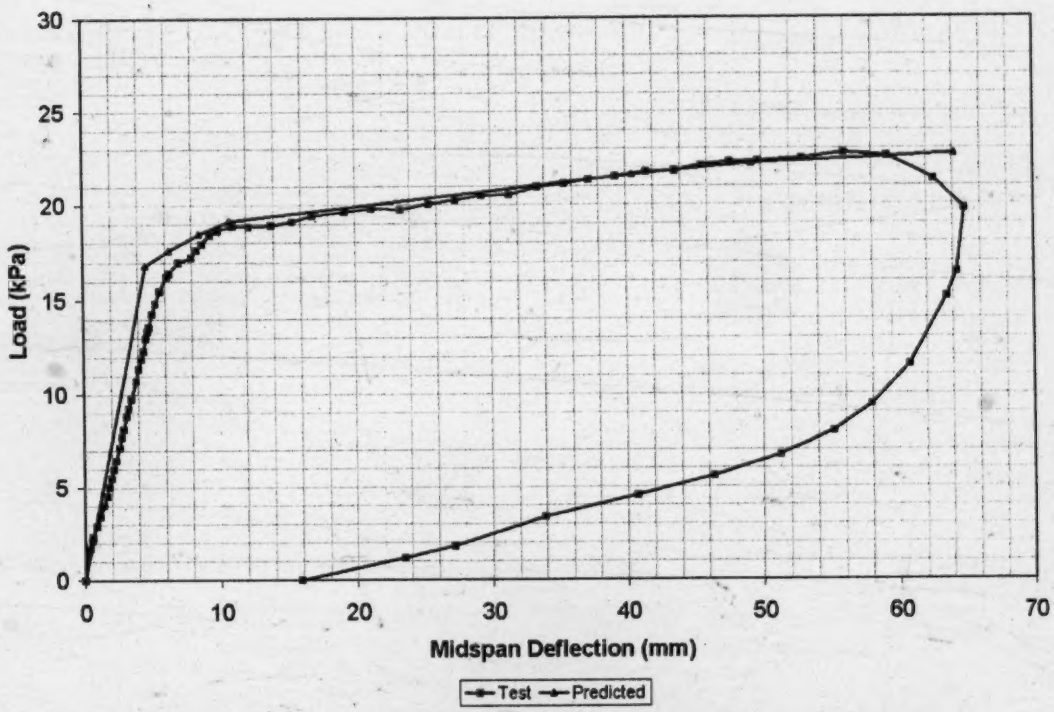

Figure 4.2 Flexural Test - Predicted Load vs. Mid-span Deflection (Test Series A2)

4.5.3 Test Series A3

Figure 4.3 shows the predicted net load vs. mid-span deflection curve along with the tested net load vs. mid-span deflection curve for test specimen A3. The predicted curve closely represents the initial stiffness of the test specimen. When the panel begins to deform plastically the predicted curve underestimates the actual capacity of the test specimen. The strip hooks and erection hooks provide additional shear transfer between the two concrete wythes. It is assumed that the additional capacity observed by the test specimen resulted from the presence of the stripping hooks, the erection hooks etc.

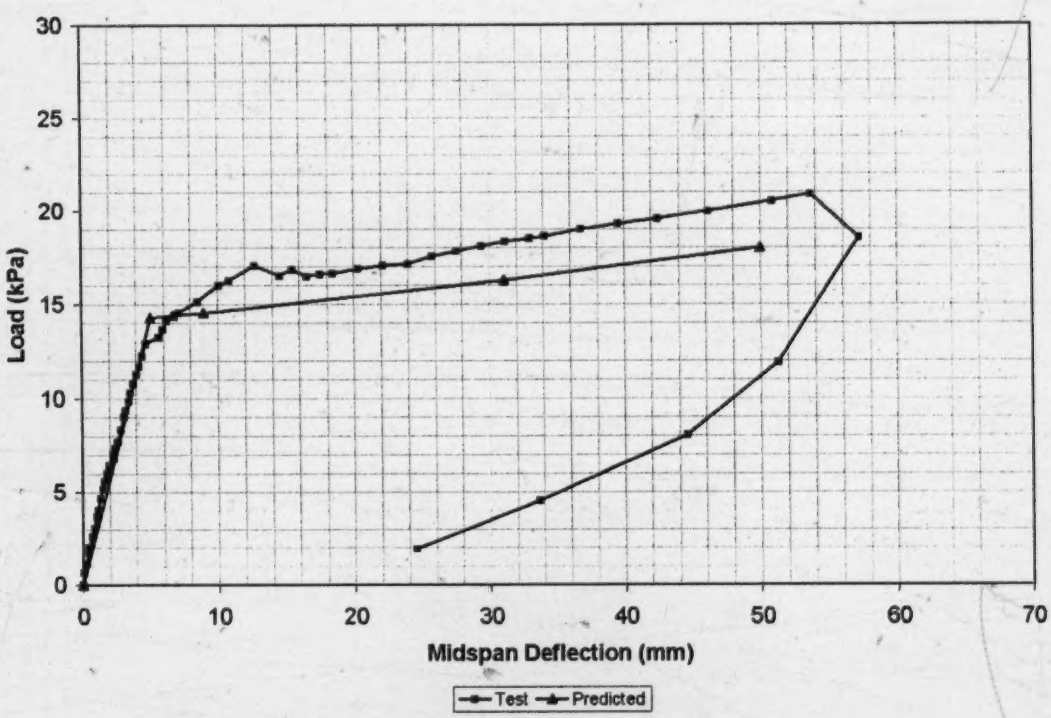

Figure 4.3 Flexural Test - Predicted Load vs. Mid-span Deflection (Test Series A3)

4.6 Discussion

Test specimens A1 and A2 used an expanded metal shear connector. These specimens experienced a higher degree of composite action with the strength-based definition of Eqn. [4.2]. The specimens were cut from much larger panels, hence the full size panels would have more shear connectors and an even higher degree of composite action. It is estimated the unfactored ultimate capacity of the full-sized panels would be between 3 and 4 kPa. This is greater than the required factored design load of 1.5 kPa for Edmonton. On the other hand, test specimen A3 used M-tie shear connectors. This specimen experienced a very low degree of composite action. The capacity of test panel A3 was 20.9 kPa, which is also greater than the design load of 1.5 kPa. The run-of-production panels appear to have very conservative designs. Walls A1 and A2 have about twice as much capacity than required for wind loads. Wall A3 is about 10 times stronger than required for wind loads.

In all cases, additional shear transfer occurs as a result of a frictional force between the insulation and the concrete wythes. Although this frictional force cannot be relied upon in design, it was used in the analysis of the specimens. It appears that test specimen A1 had a significant bond between the insulation and the concrete during testing resulting in a higher than predicted ultimate bending capacity. After 35 mm of wall deflection the bond finally broke down.

Assuming the bond between the insulation and concrete has failed and there are no stripping or erection details present, the method used to predict the overall behaviour of the test specimens appears to be reasonably accurate and on the conservative side. Again, if the bond between the insulation and the concrete is broken down and there are no stripping or erection details present it appears as though the simplified ultimate capacity analysis estimates the ultimate bending capacity conservatively and with reasonable precision. Typically the bending moments of the individual concrete wythes is equal to or slightly greater than the cracking moment when the shear connectors begin to fail. The simplified analysis assumes the bending moment capacity of the individual concrete wythes to be equal to the cracking moment.

4.7 Conclusion

From the preliminary experimental program:

- A rational analytical model for predicting behaviour is presented.
- The characteristics of some shear connectors were established to allow for rational analytical analysis.
- The production panels used for the preliminary testing program appear to have excess capacity.
- It is proposed that a full size wall be designed using the developed theory to test its validity.

5 Full-Scale Experimental Program

5.1 Overview of Full-Scale Experimental Program

The objective of the full-scale experimental program was to test the validity of the analysis and design procedures from Chapter 4. The procedures suggest that acceptable panels can be produced with fewer strands and fewer shear connectors than used in current design. A full sized panel was designed using the procedure discussed in Chapter 4 for all applicable load cases including; stripping and handling, service wind loads, and ultimate wind loads. The panel was constructed and tested in flexure. Ancillary tests include: unit shear tests; tension coupon tests cut from the shear connectors; friction tests between the insulation and polyethylene bond-breaker; and concrete compression tests. The details and materials used in the full size wall panel were controlled or measured to a degree not possible with the preliminary test series.

5.2 Flexural Tests

The flexural test specimen was designed as a standard 2.385 m wide by 9.754 m tall sandwich panel for a service wind load of 1.0 kPa, and a span of 9.144 m. The panel contained a 100 mm (4") thickness of insulation. The design calculations for the test specimen are shown in Appendix C. All the necessary load and resistance factors were applied during the calculations. The test specimen was designed for a factored design load of 1.5 kPa.

Figure 5.1 shows the as-built drawing of flexural test specimen B1. The test panel was cast in the same bed as another panel requiring more strands than specified for the test panel. As a result it was necessary to de-bond two strands in the compression wythe and three strands in the tension wythe. The test panel therefore had four fully bonded 3/8" strands in each wythe as specified. The sandwich panel material properties are summarized in Table 5.1. To estimate the effective stress in the strands at the time of testing the detailed procedure outlined in the CPCI Design Manual (CPCI, 1996) used to calculate prestressing loss was used. The test specimen was cast 15 days prior to testing. Calculations showed that four strands per wythe after all long term losses were more than

sufficient to meet the minimum requirement of 1 MPa average compression stress in the concrete due to prestressing.

The type of insulation used in the test panel was the Deckmate insulation produced by DOW Chemical Canada Inc. A polyethylene bond-breaker was placed between the insulation and the concrete to simulate the loss of bond – a worst case scenario for panel behaviour. Expanded metal shear connectors as shown in Figure 5.2 were used. From Figure 5.3 it can be seen that the shear connectors fit neatly between the gaps of the insulation sheets resulting in improved constructability compared to many styles of shear connectors. For handling purposes four lift hooks were provided. Prior to testing all lift hooks were cut so they would not provide any additional flexural capacity to the test specimen.

The density of the concrete and insulation were also measured. The density of the concrete was found to be 2232.7 kg/m^3 and the density of the insulation was found to be 28.0 kg/m^3. The overall weight of the panel was determined to be 8067 kg.

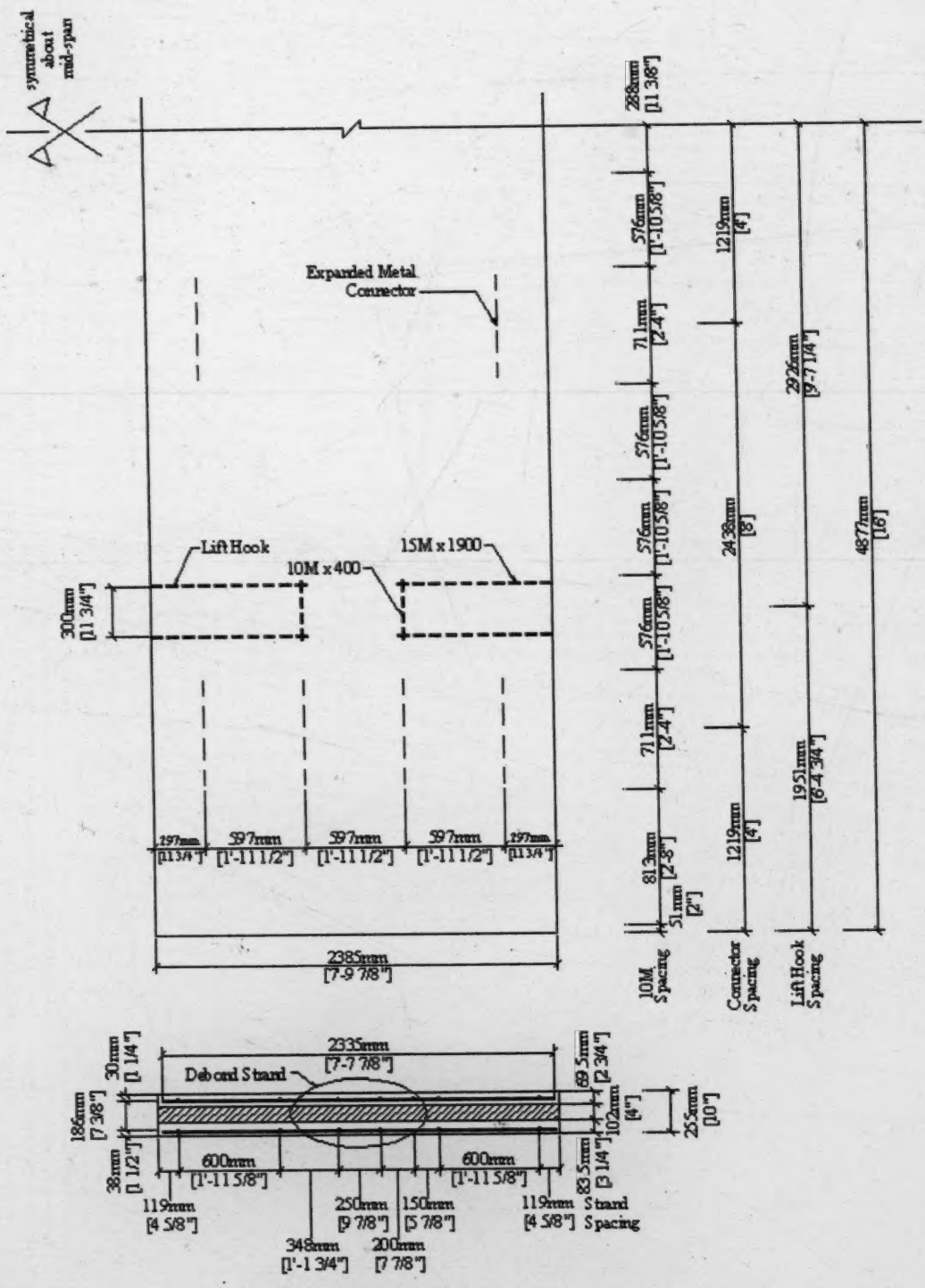

Figure 5.1 Full-Scale Flexural Test Specimen B1

Table 5.1 Full-Scale Sandwich Panel Material Properties

	Test Specimen B1
Compression Wythe	
Concrete Properties	
f'_c at time of test	43.2 MPa
Strand Properties	
# of Strands	4
Strand Diameter	3/8"
f_{pu}	1860 MPa
Stressed to before losses	0.70 f_{pu}
Stressed to after losses	0.65 f_{pu}
Tension Wythe	
Concrete Properties	
f'_c at time of test	41.2 MPa
Strand Properties	
# of Strands	4
Strand Diameter	3/8"
f_{pu}	1860 MPa
Stressed to before losses	0.70 f_{pu}
Stressed to after losses	0.65 f_{pu}
Connector Properties	
Type	Expanded Metal
# of Connectors	12
Insulation Properties	
Type	Deckmate
Thickness	4"

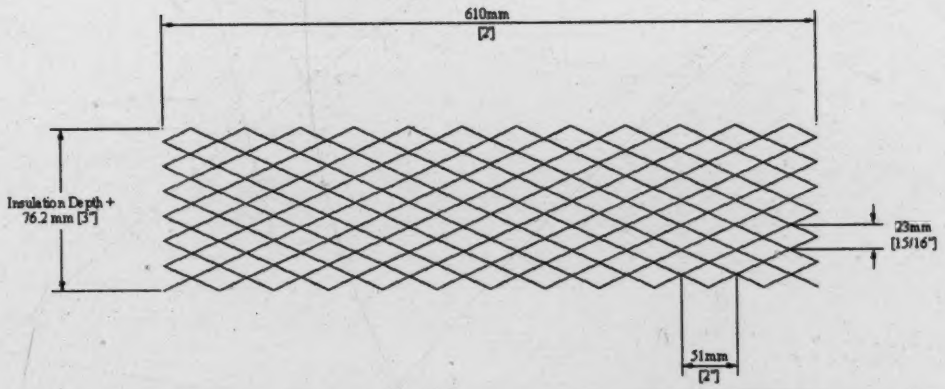

Figure 5.2 3/4"-#13 Galvanized Steel Standard Expanded Metal Shear Connector

Figure 5.3 Full-Scale Flexural Test Specimen B1 (Construction)

The flexural test set-up used for the full-scale test was very similar to the test set-up used in the preliminary experimental program. Due to the size of the test specimen it was necessary to perform the full-scale flexural test at the Lafarge Canada Inc. precast plant in Edmonton. The test specimen was just under 8' (2400 mm) wide and was tested at a 30' (9144 mm) span. Once again the panel was tested in the horizontal position. A series of water bags were used to apply a uniformly distributed upward load (Figure 5.4). The test set-up is shown in Figure 5.5 to Figure 5.7. Figure 5.6 shows the test panel being handled in the horizontal position about to be placed into the test frame. The flexural test specimen was placed above the water bags and rested on 4x4 supports. There were 4x4 supports located at each end of the panel. These 4x4 supports were bolted to the concrete pad to prevent the water bags from bulging out the ends of the test set-up. There were also two intermediate supports so the test specimen did not fail under its self-weight. To prevent the water bags from bulging out the side of the test set-up concrete "lock-blocks" (large concrete blocks) held plywood sheets in place as a restraint. Round HSS 114x8 sections were used as the end supports to provide simply supported boundary conditions. The HSS was anchored to the concrete pad.

To measure the pressure in the water bags a standpipe was used. Various corrections were used to adjust the standpipe readings to represent the applied pressure on the panel. Corrections were made to account for the self-weight of the panel and the piezometric head loss under the panel as the panel deflected upward. The end reactions were also measured with the use of four load cells placed above the reaction supports on the threaded rods. The load cell readings were corrected by adding the weight of the support apparatus and the weight of the wall panel projecting beyond the support centre lines. The load cell readings were then used to verify the applied pressure calculated from the standpipe readings. Two cable transducers were used to measure the vertical mid-span deflection. The vertical displacement of the end supports was also measured using LVDT's in order to calculate the relative mid-span deflection. Dial gauges were used to measure the slip between the two concrete wythes at each end of the panel.

The results of the flexural tests are presented along with the results of the ancillary tests.

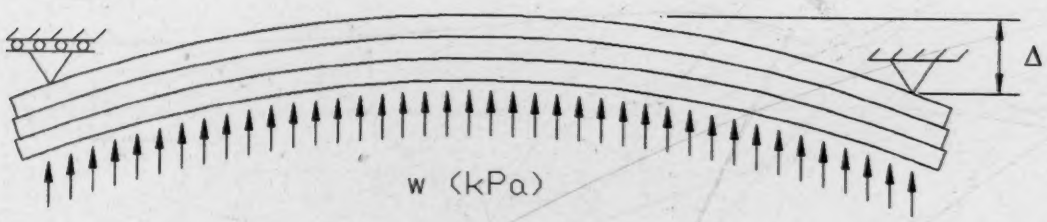

Figure 5.4 Flexural Test Schematic

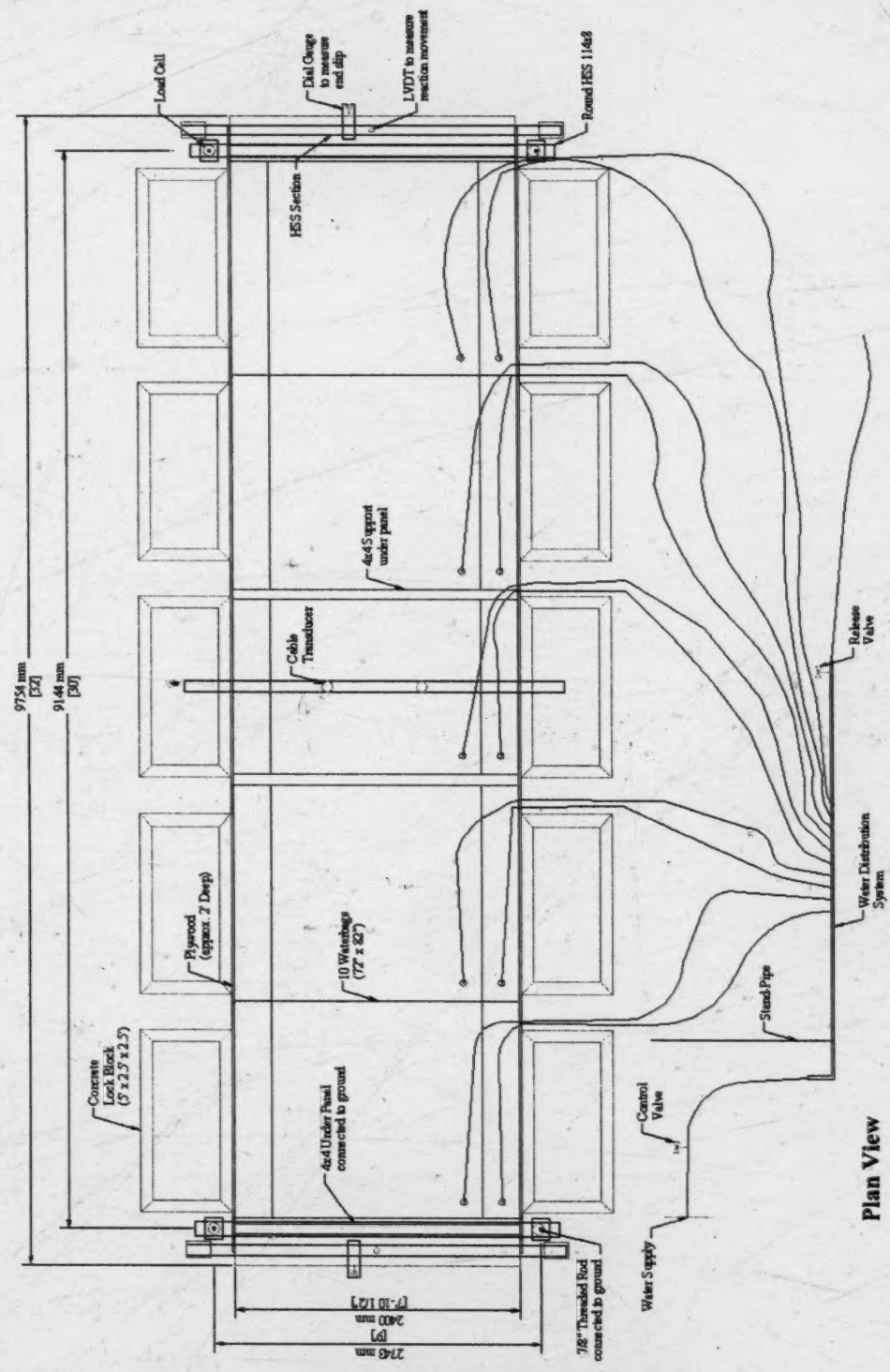

Figure 5.5 Full-Scale Flexural Test Set-up (Drawing)

Figure 5.6 Full-Scale Flexural Test Set-up (Water Bags)

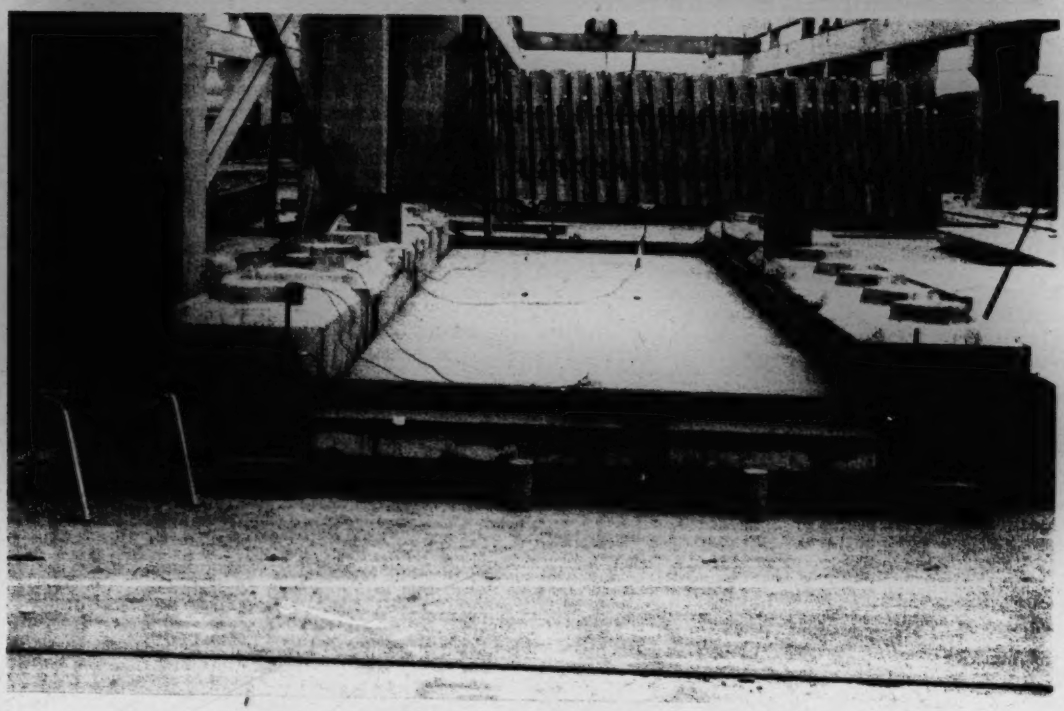

Figure 5.7 Full-Scale Flexural Test Set-up (Picture)

5.3 Ancillary Tests

This section describes the test procedures and test specimens for the ancillary tests performed to characterize the properties of the individual sandwich panel components. The ancillary tests performed include; unit shear tests, coefficient of friction tests, tension coupon tests, and concrete compression tests.

5.3.1 Unit Shear Tests

Unit shear tests were performed to characterize the behaviour of the shear connectors used in the full-scale flexural test specimen. The most common insulation dimensions used in precast sandwich panels are 2", 3", and 4". It was found from the previous unit shear tests that the specimens using the thicker insulations result in a lower shear connector ultimate capacity. Thus a conservative shear connector capacity was obtained from the shear test specimens by using a 4" insulation thickness.

A typical unit shear test specimen is shown in Figure 5.8. The style of expanded metal shear connector used was a galvanized steel 3/4"-#13 standard expanded metal. The unit shear test specimens contained a polyethylene bond-breaker between the insulation and the concrete so the characteristics of the shear connectors could be measured.

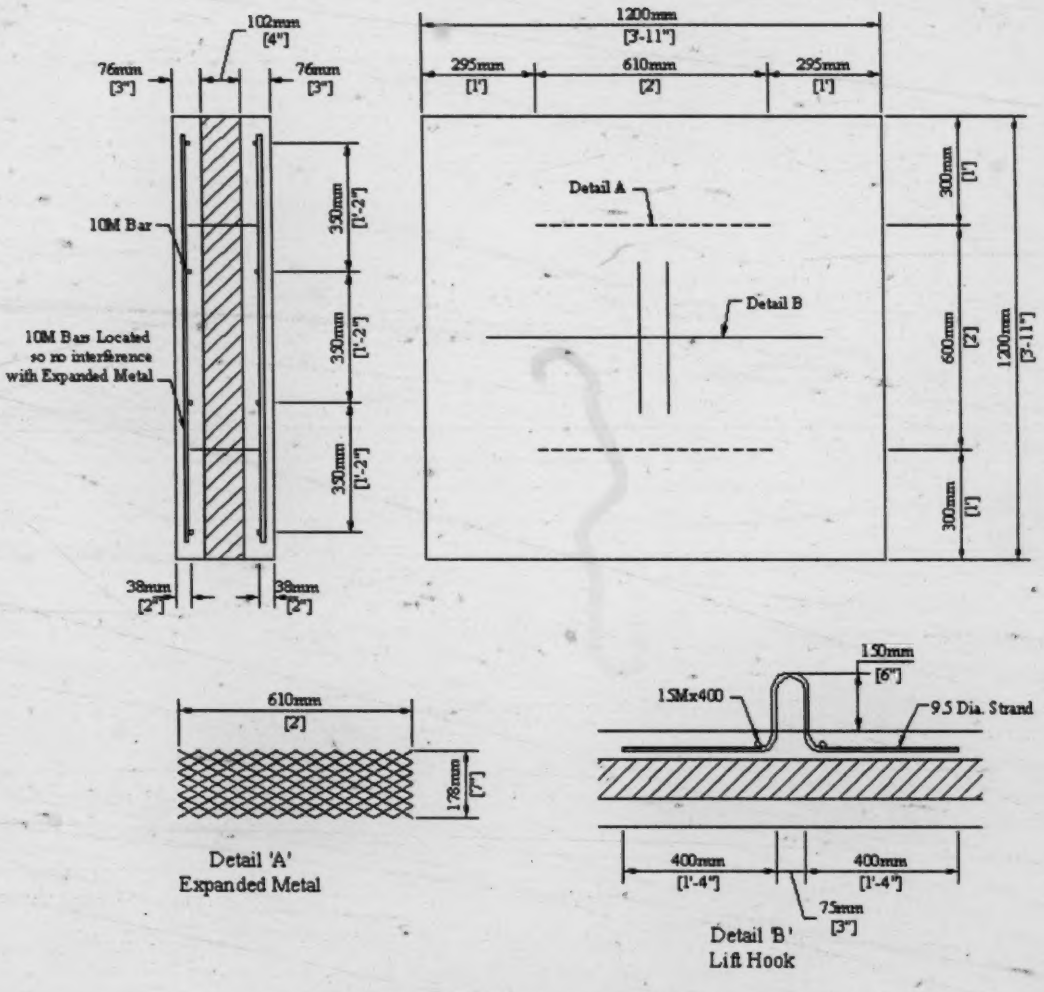

Figure 5.8 Full-Scale Unit Shear Test Specimen

To perform the unit shear tests, the shear connectors are put into direct shear by pushing the top wythe of concrete relative to the bottom wythe of concrete (Figure 5.9). This was done in the test frame shown in Figure 5.10 and Figure 5.11. The test frame was designed and built for the shear tests and for the purposes of quality control in the future. During the test the top wythe of concrete was pushed relative to the bottom wythe using a jack. The applied jacking force was measured with the use of a load cell. To prevent the test specimen from overturning a set of double-channels was used to hold down the test specimen near the jacking force. A load cell was placed under the double-channels to measure the uplift force. The uplift force was measured so the frictional force between the insulation and concrete could be determined. The frictional force was

subtracted from the jacking force to determine the shear force experienced by the connectors. The relative displacement between the top and bottom wythes of concrete was measured with two dial gauges. From the test data a load vs. displacement curve was developed.

The results of the unit shear tests are reported with the results from the other tests.

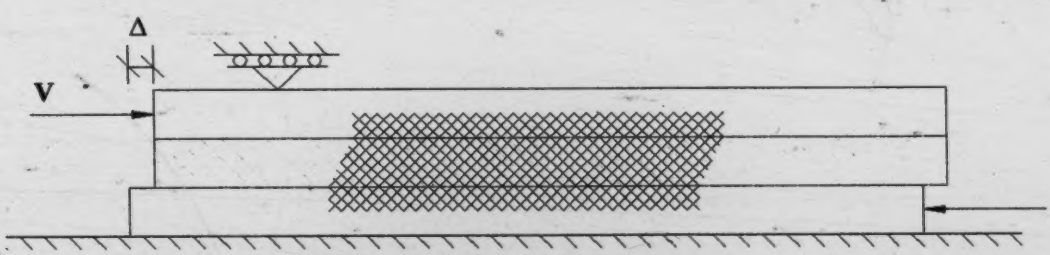

Figure 5.9 Unit Shear Test Schematic

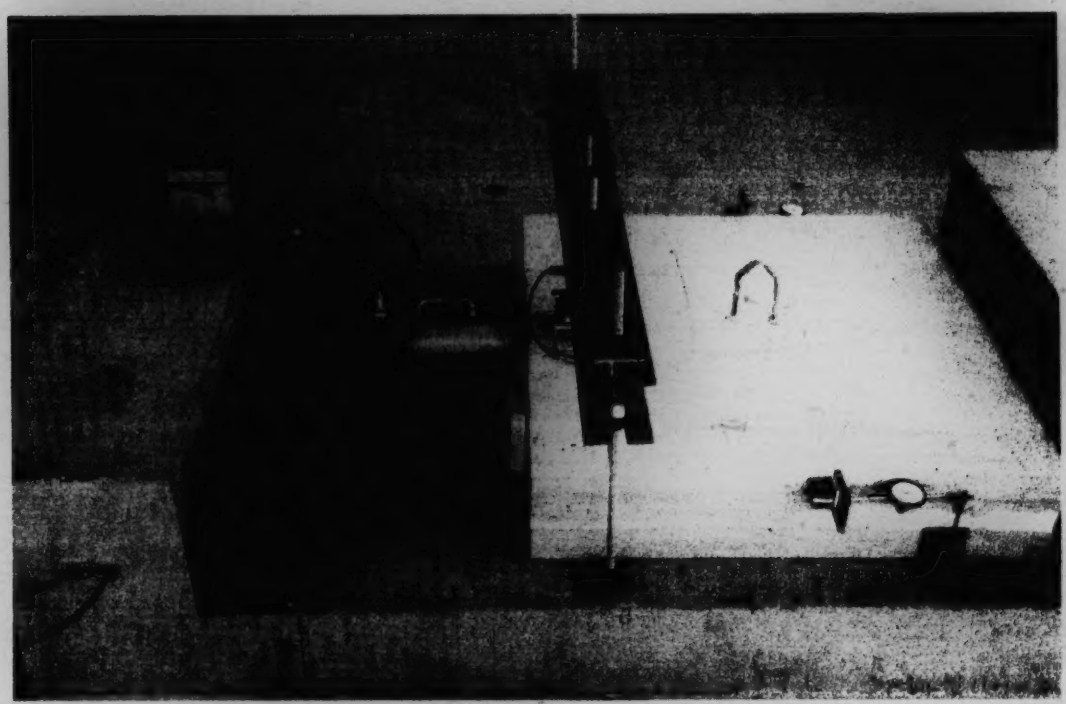

Figure 5.10 Full-Scale Unit Shear Test Set-up (Picture)

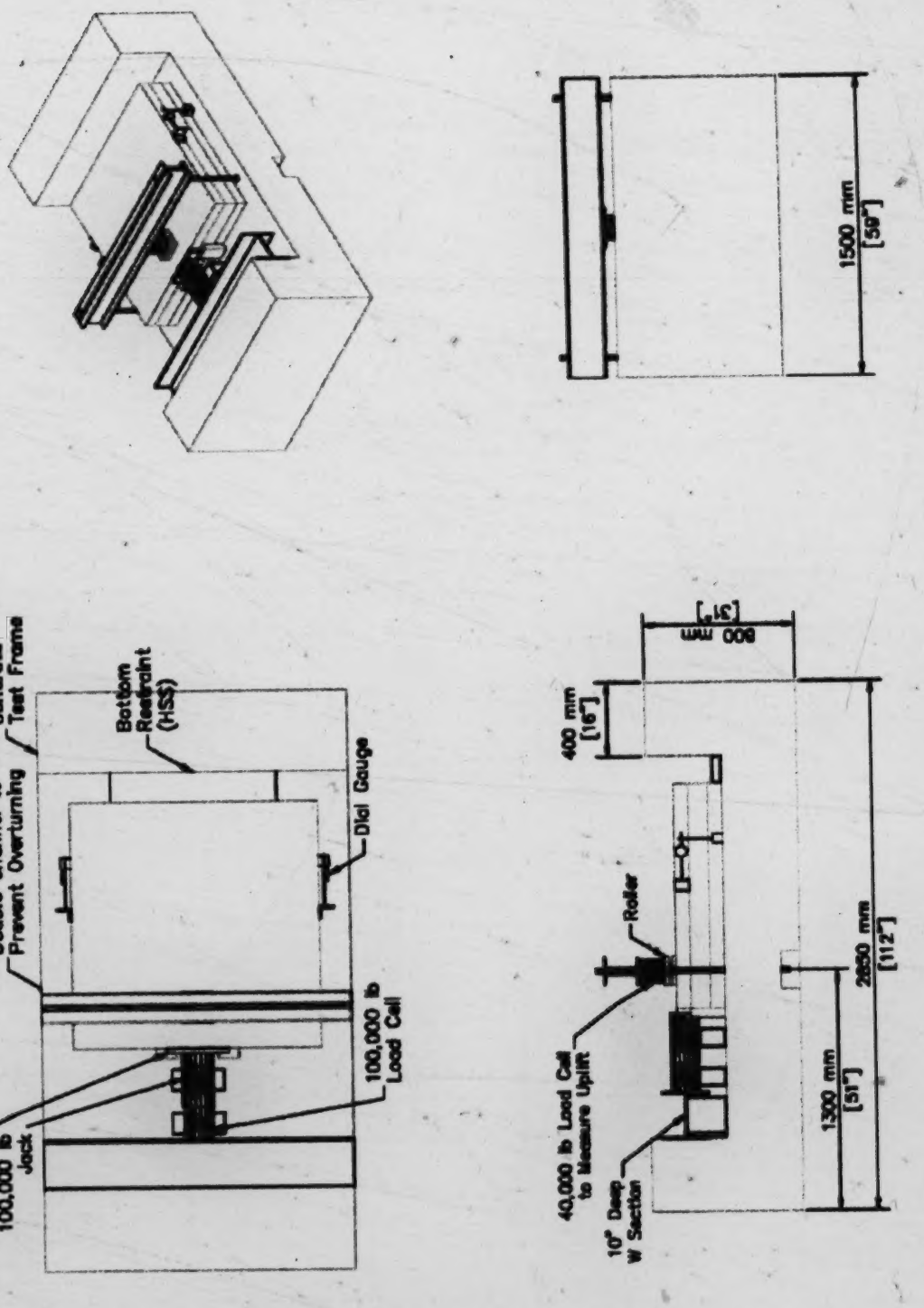

Figure 5.11 Full-Scale Unit Shear Test Set-up (Drawing)

5.3.2 Coefficient of Friction Tests

Tests were performed to determine the coefficient of friction between the insulation and the polyethylene bond-breaker. The coefficient of friction is required for the analysis of the flexural test specimen, which also had a polyethylene bond-breaker present. Friction between the insulation and concrete allows for increased shear transfer between the two wythes. Friction results in a slight increase in composite action.

The test specimens were 400 mm wide and 932 mm long. The tests were performed by tilting the test specimens until slip occurred between the insulation and the concrete. The angle between the test specimen and the horizontal was measured. To determine the coefficient of friction Eqn. [3.2] is used. The frictional force can then be calculated using Eqn. [3.1]. The results of the coefficient of friction tests are reported with the results of the other tests.

5.3.3 Tension Coupon Tests

Tension tests were performed on a diagonal cut from a section of expanded metal to characterize the material properties of the expanded metal shear connectors (Figure 5.12). The tests were performed in the universal testing machine shown in Figure 5.13. Measurements taken include the machine load and stroke along with the strain measured using the clip-on extensometer visible in Figure 5.13. The dimensions of the expanded metal were also measured prior to loading so the stress in the metal could be calculated. The results of the tension coupon tests are reported with the results of the other tests.

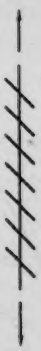

Figure 5.12 Tension Coupon Test Schematic

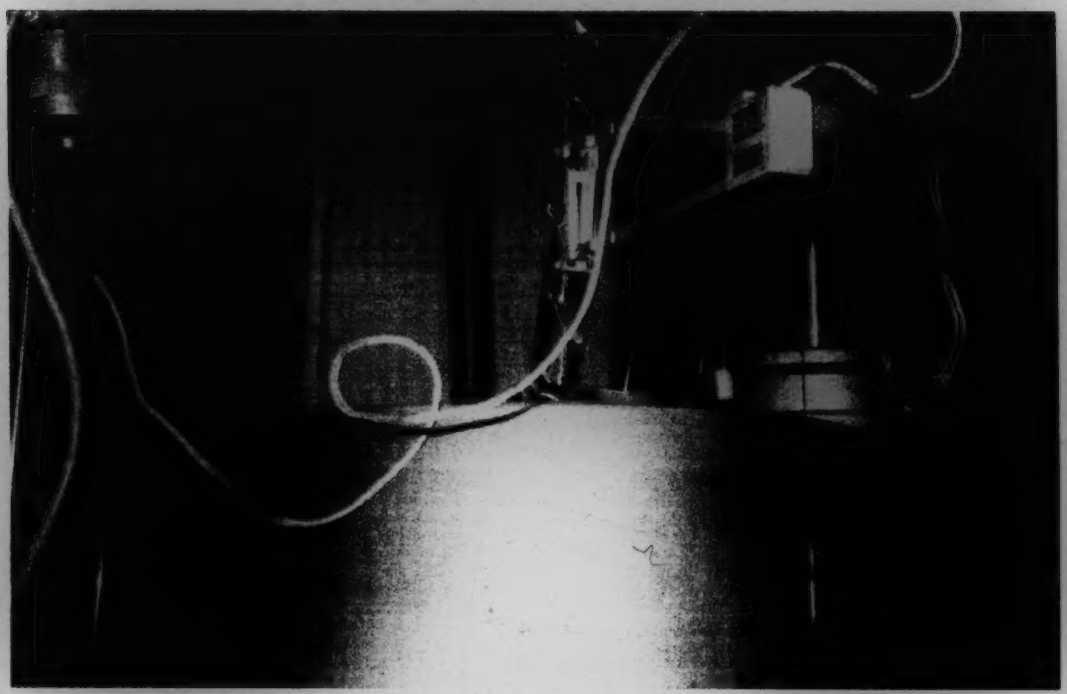

Figure 5.13 Full-Scale Tension Coupon Test

5.3.4 Concrete Compression Tests

The concrete compressive strengths for the flexural tests were obtained from compression tests performed on cylinders cast and cured alongside the full-scale flexural test specimen. The test cylinders had a 4" diameter and were tested at the time the full-scale flexural test was performed. The concrete strengths are reported in Table 5.1.

5.4 Test Results & Observations

This section presents the results from the full-scale flexural tests, unit shear tests, tension coupon tests, and coefficient of friction tests. The full-scale flexural test results are presented as a transverse load vs. mid-span deflection curve and an end slip vs. mid-span deflection curve. After all corrections were applied the pressure calculated from the load cells agreed within 5% of the pressure calculated from the standpipe readings. The average pressure calculated from the load cells and the standpipe was used in this section. A picture of the crack pattern that developed on the tension face of the flexural test specimen is also provided.

The unit shear test results are presented as a shear load per connector vs. shear displacement curve. The tension coupon tests are presented as a stress vs. strain diagram. The clip-on extensometer had difficulty gripping the specimen. The strain calculated using the stroke of the testing machine provided more consistent results than the extensometer. Therefore the strain calculated from the machine stroke is presented in this section.

5.4.1 Test Series B1

Despite the fact a polyethylene bond-breaker was used in the full-scale flexural test specimen, a significant "bond" appeared to exist between the insulation and the concrete. Upon loading the test panel appeared to be much stiffer and much stronger than anticipated. The test specimen was loaded gradually until cracking occurred and the bond between the insulation and the concrete was broken. This occurred at a load of 3.87 kPa. At this time, the panel was unloaded. A fully composite panel would have had an ultimate capacity of 4.03 kPa therefore it is apparent the test panel was behaving as a fully composite panel. Test specimen B1 was then reloaded. The net load vs. mid-span deflection curve for the second loading is shown in Figure 5.14. The test specimen remained virtually linear elastic up to a deflection of 64.6 mm. The test specimen then maintained its load and deformed plastically. Prior to stopping the test, the test specimen obtained an ultimate capacity of 2.32 kPa and underwent a deformation of 179.3 mm. It was decided to stop the test because the specimen had demonstrated more than enough ductility.

Figure 5.15 shows the end slip vs. mid-span deflection for flexural test specimen B1. It can be seen that the shear connectors on either side of the panel underwent similar deformations up to a mid-span deflection of 50 mm. At this time the connectors on the south half of the panel began to fail and underwent large deformations. The south half of the panel had a maximum end slip of 15.79 mm prior to stopping the test.

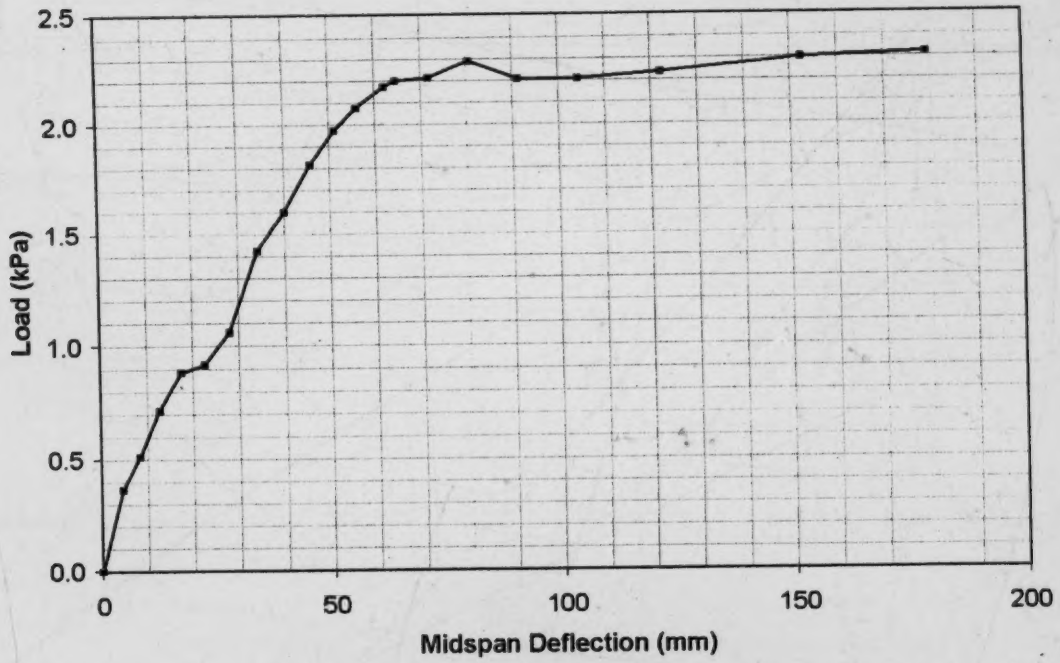

Figure 5.14 Flexural Test - Load vs. Mid-span Deflection (Test Series B1)

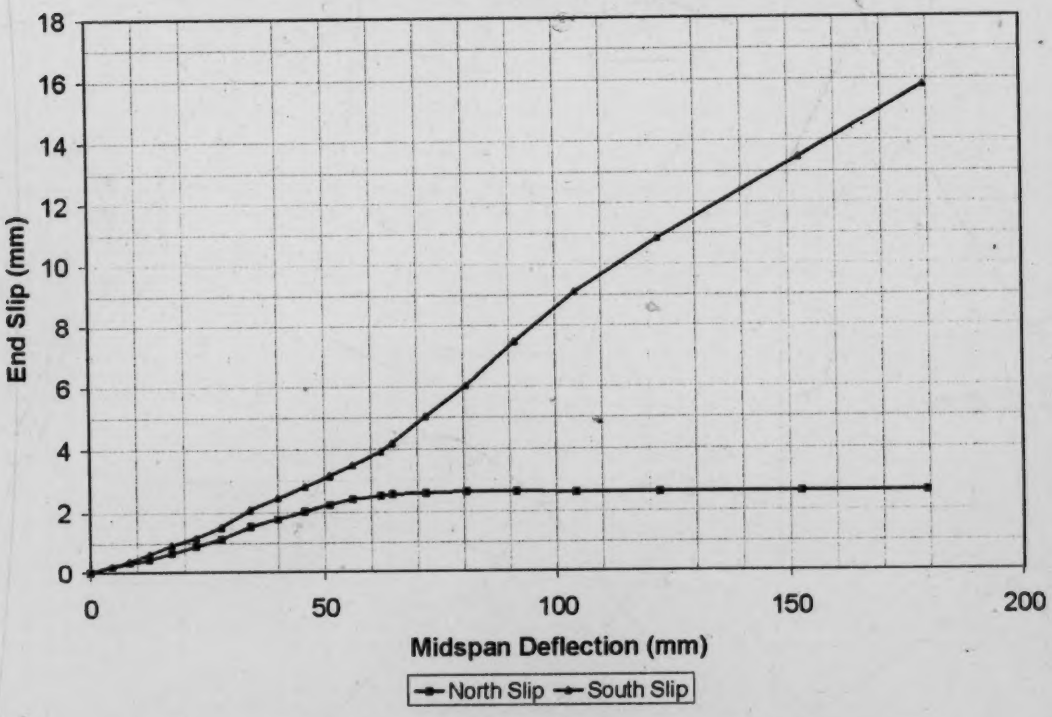

Figure 5.15 Flexural Test - End Slip vs. Mid-span Deflection (Test Series B1)

The crack pattern that developed on flexural test specimen B1 is shown in Figure 5.16. Crack 1 was the primary flexural crack. The crack formed just outside the location of the inner shear connectors on the south half of the panel. When the test was finally finished, the crack was open approximately 2 mm. The next crack developed at mid-span. When the test was completed the crack was approximately 1 mm wide. The last crack to form occurred just outside of the inner shear connectors on the north half of the panel. The cracks labelled 3 were less than 1 mm wide when the test was brought to completion.

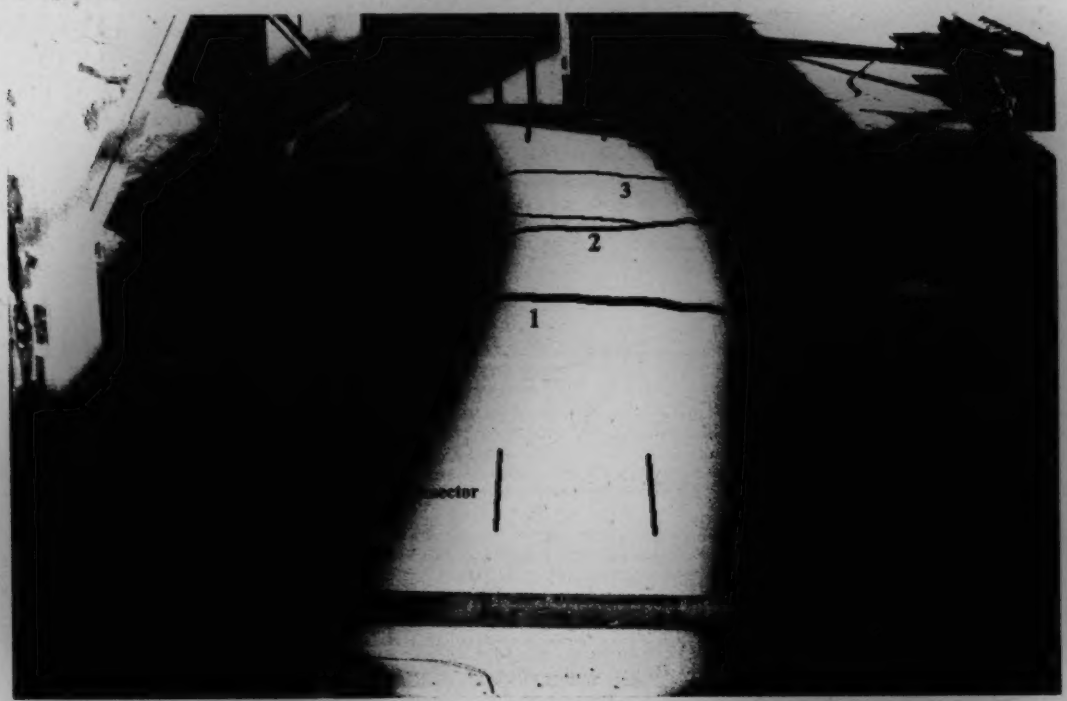

Figure 5.16 Crack Pattern (Test Series B1)

Three unit shear test specimens were fabricated with the galvanized 3/4"-#13 standard expanded metal connectors. Figure 5.17 shows the load vs. displacement curves for the unit shear tests. Test specimen B1-1 appeared to have a lower initial stiffness than the other two test specimens. Test specimen B1-1 remained linearly elastic up to a load of 18.4 kN/connector and a displacement of 1.43 mm. At this point the specimen began to plastically deform and reached an ultimate capacity of 23.9 kN/connector before the specimen finally failed at a displacement of 8.01 mm. Test specimen B1-2 remained

linearly elastic up to a load of 23.5 kN/connector and a displacement of 0.30 mm. The specimen then began to plastically deform and reached an ultimate capacity of 26.9 kN/connector before the specimen failed at a displacement of 6.43 mm. Test specimen B1-3 remained linearly elastic up to a load of 20.3 kN/connector and a displacement of 0.36 mm. The test specimen then plastically deformed and reached an ultimate capacity of 23.9 kN/connector before the connectors failed at a displacement of 6.71 mm.

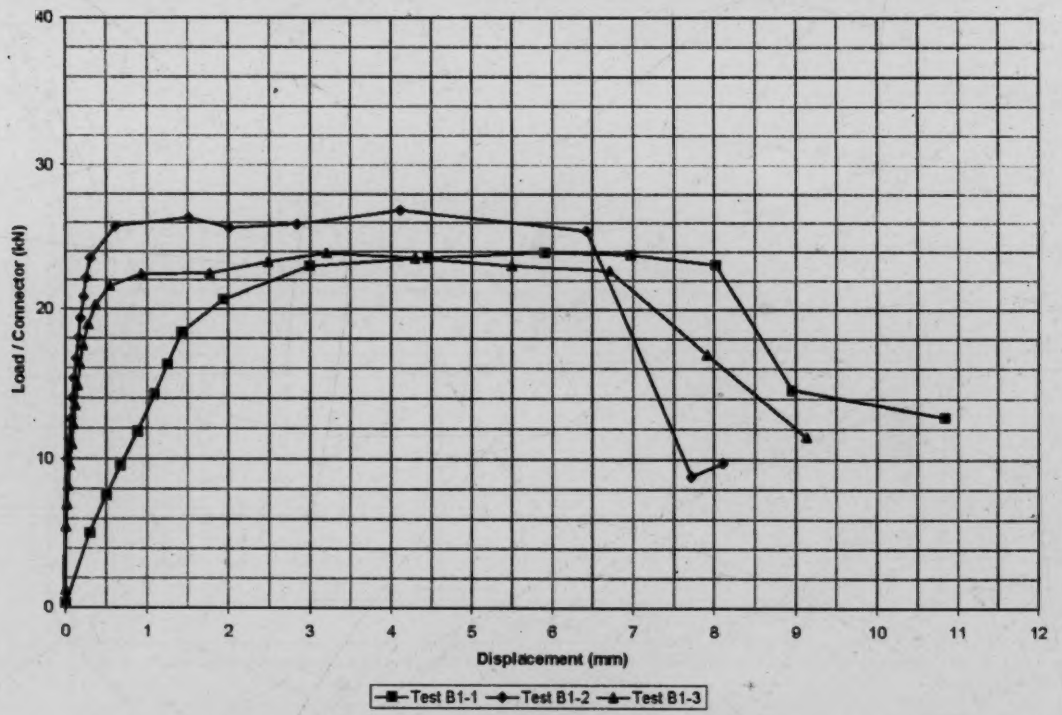

Figure 5.17 Unit Shear Test - Load vs. Displacement (Test Series B1)

Three tension coupon tests were performed on specimens cut from galvanized steel 3/4"-#13 standard expanded metal. The gauge lengths of the test specimens were all 252 mm. The test results (Figure 5.18) proved to be very consistent. The spikes in the graph represent static readings; therefore the approximate static ultimate capacity is 335 MPa at a strain of 40,600 με. The expanded metal specimen exhibited a ductile bilinear response.

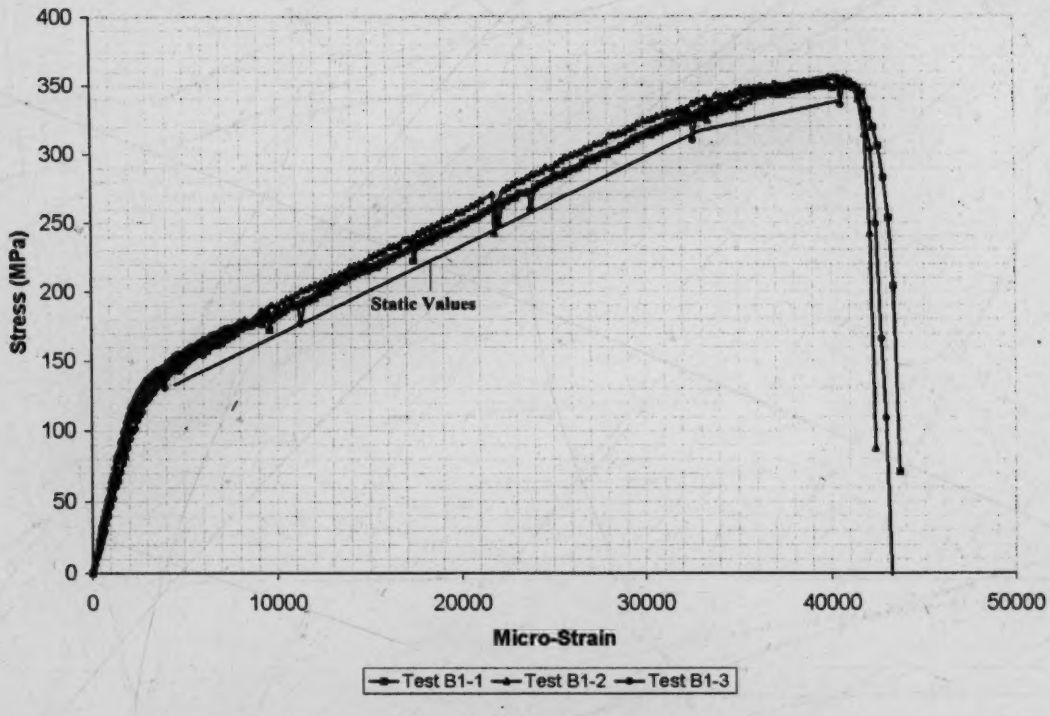

Figure 5.18 Tension Coupon Test - Stress vs. Strain (Test Series B1)

To calculate the stress from the testing machine load it was necessary to measure the cross-sectional properties of the expanded metal. A definition of the cross-sectional properties is provided in Figure 5.19 (ASTM, 1997). The average width of mesh (a) was determined to be 23.68 mm, the average length of mesh (b) was determined to be 50.74 mm, the average width of strand (c) was determined to be 2.11 mm, and the average thickness of strand (d) was determined to be 2.72 mm. The cross sectional area of the strand is then calculated to be 5.72 mm^2.

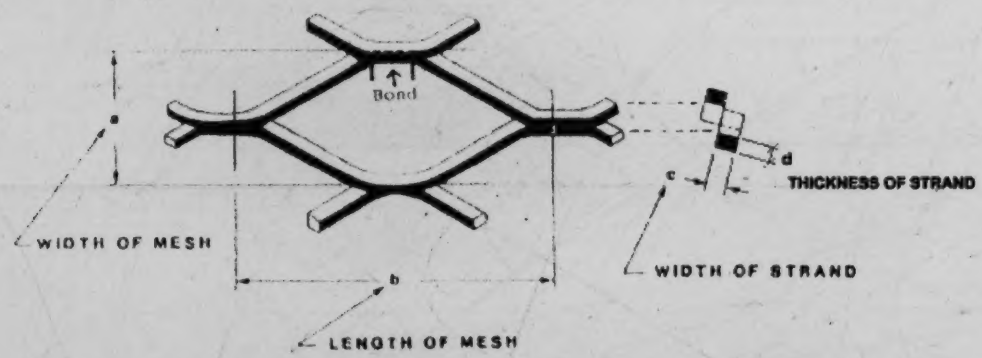

Figure 5.19 Definition of Expanded Metal Cross-Sectional Properties

The coefficient of friction between the insulation and the concrete when a bond-breaker is present is required for analysis purposes. The polyethylene remained bonded to the concrete. Slip occurred between the insulation and the polyethylene at an average angle of 29.5° between the specimen and the horizontal. The coefficient of friction is then calculated to be 0.57.

5.4.2 General Observations

Flexural test specimen B1 failed in a ductile manner. The test specimen exhibited a semi-composite behaviour with failure occurring in the shear connectors.

From the unit shear tests it appears that the expanded metal shear connectors have some characteristics that are beneficial for use in partially composite panels. The expanded metal connectors nearly reach their ultimate capacity with little permanent deformation and then maintain this capacity for a reasonably large deformation. The connectors reached an average load of 21 kN/connector with little permanent deformation and then increased in capacity up to 25 kN/connector prior to failing at an average shear deformation of 7 mm.

The coefficient of friction between concrete and the Deckmate insulation is not significantly different if there is a polyethylene bond-breaker present or not. The average coefficient of friction determined during the preliminary experimental program between concrete and the blue styrofoam brand insulation was 0.61. When a bond-breaker was present the coefficient of friction was determined to be 0.57.

5.5 Evaluation of Results

Table 5.2 summarizes the predicted flexural capacity calculated using the simplified analysis procedure along with the calculated degree of composite action for flexural specimen B1. The analysis procedures described in Chapter 4 were used in this section.

Table 5.2 Evaluation of Full-Scale Test Results

	Test Specimen B1
Loads	
W_{test}	2.32 kPa
$W_{predicted\ (Overall\ Behaviour\ Anal)}$	2.22 kPa
W_{test} / $W_{predicted\ (Overall\ Behav.)}$	1.045
$W_{predicted\ (Simplified\ Analysis)}$	2.25 kPa
W_{test} / $W_{predicted\ (Simplified)}$	1.030
% Composite	
Eqn. [2.4]	4.9 %
Eqn. [4.2]	
excluding V_{insul}	37.4 %
including V_{insul}	43.7 %

Figure 5.20 provides the tested net load vs. mid-span deflection curve along with the predicted net load vs. mid-span deflection curve for flexural test specimen B1. The predicted curve was developed using the same procedures documented in Chapter 4 and Appendix A. There is a slight discrepancy near the end of the curve when the test curve maintains its capacity while the prediction curve slightly decreases in capacity.

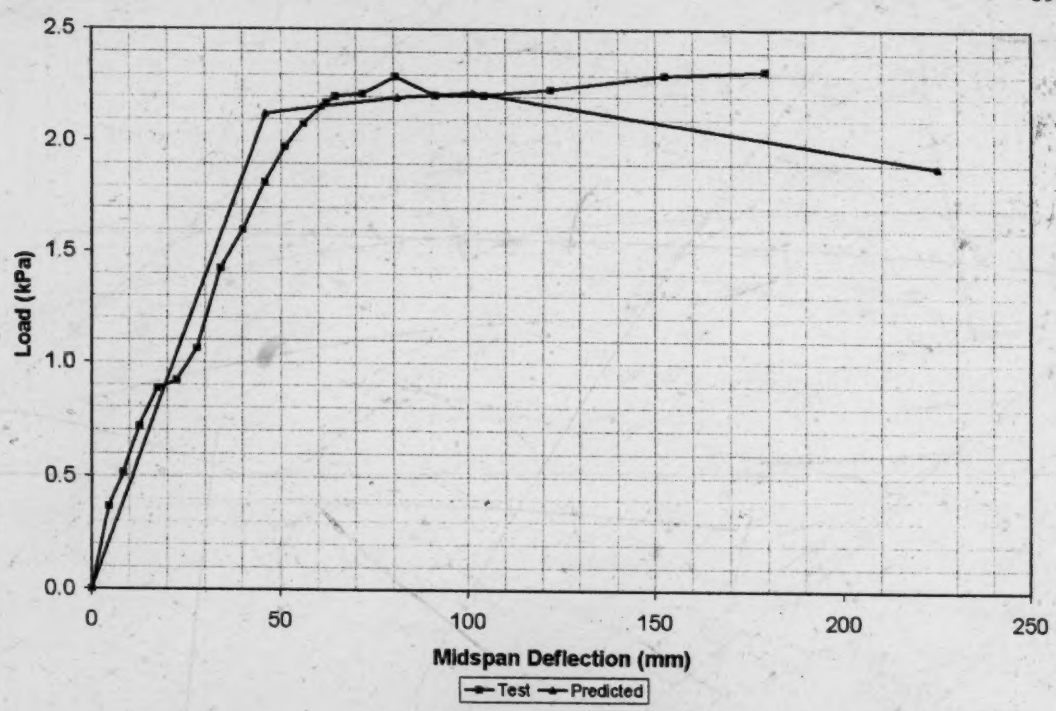

Figure 5.20 Flexural Test - Predicted Load vs. Mid-span Deflection (Test Series B1)

5.6 Discussion

Current designs for sandwich panels with dimensions and loads selected for the full-scale test would have five to seven strands per wythe and 24 to 32 shear connectors. The test specimen, which was designed with the procedure outlined in Chapter 4, used four strands per wythe and 12 shear connectors. The test specimen had a mid-span deflection of L/350 at a service load of 1 kPa. The allowable deflection depends on the use of the building but would typically be between L/240 and L/180. Thus, the test specimen would satisfy serviceability limit state requirements. The test specimen developed an ultimate capacity of 2.32 kPa, which is greater than the factored load of 1.5 kPa. Thus, the test specimen would satisfy ultimate limit state requirements.

5.7 Conclusion

The analysis procedures developed during the analysis of the preliminary experimental program predicted the behaviour of the full-scale test specimen with reasonable accuracy. The ultimate bending capacity was predicted within 3% using the simplified analysis procedure outline in Appendix B. As a result of the testing and analysis of the full-scale sandwich panel it was concluded that panels using fewer prestressing strands and fewer shear connectors than traditional sandwich panels can successfully be designed to resist the required design loads.

6 Conclusions & Recommendations

Flexural tests were performed on partially composite concrete sandwich panels. In addition to the flexural tests, ancillary tests were also performed to characterize the behaviour of the individual sandwich panel components. The results of the testing program were used to develop and verify a model to analyse and design sandwich panels.

The behaviour of partially composite sandwich panels is comparable to the behaviour of coupled shear walls. The concrete wythes act like the walls while the wythe connectors act like the coupling beams. Shear is transferred through the wythe connectors causing the two concrete wythes to act together to resist loads. More specifically, the shear transfer creates tension in one wythe and compression in the other wythe. The moment arm between these tension and compression force creates additional moment resistance for the sandwich panel. The degree of composite action is dependent on the style, location, and number of connectors used to tie the wythes together. Many definitions exist to define the degree of composite action. The definition provided in this thesis is similar to the definition of composite action provided in CSA S16.1-94. The degree of composite action is defined as follows:

$$\%\ Composite = \frac{(\phi_{sc} \times V_{sc}) + (\phi_{insul} \times V_{insul})}{\phi_p A_p f_{pr(tension\ wythe)}} \qquad [4.2]$$

but not greater than 100%

Where: ϕ_{sc} = resistance factor for the shear connector
V_{sc} = the load carried by the shear connectors between locations of maximum moment and zero moment (i.e. half of panel)
ϕ_{insul} = resistance factor for the shear connector
V_{insul} = shear transferred through the insulation between locations of maximum moment and zero moment (i.e. half of panel)
ϕ_p = resistance factor for the prestressing strand
A_p = area of prestressing steel in tension wythe
f_{pr} = stress in the prestressed reinforcement in the tension wythe

The above equation also considers shear that is transferred through the insulation. A significant bond forms between the insulation and the concrete. This bond results in a high degree of composite action that designers commonly rely upon for stripping,

handling, and erection loads. With time the bond likely breaks down and is therefore not relied upon for in-service and ultimate loading. Although the bond may break down, shear can still be transferred through the insulation because a frictional force exists between the insulation and concrete. This frictional force was considered for analysis of behaviour, but not considered during design.

To design and analyse the sandwich panel test specimens the fundamental equation used for coupled shear walls was slightly modified. The equation is as follows:

$$M = M_1 + M_2 + (\phi_{sc} \times V_{sc} + \phi_{insul} \times V_{insul}) \times jd \qquad [4.1]$$

Where: M_1 = the moment capacity of the compression wythe (including appropriate resistance factors for design)

M_2 = the moment capacity of the tension wythe (including appropriate resistance factors for design)

V_{sc} = the load carried by the shear connectors between locations of maximum moment and zero moment (i.e. half of panel)

V_{insul} = the shear transferred through the insulation between locations of maximum moment and zero moment (i.e. half of panel)

ϕ_{sc} = resistance factor for the shear connectors (1.0 for analysis and 0.65 for design)

ϕ_{insul} = resistance factor for the insulation friction (1.0 for analysis and 0.0 for design)

jd = distance between the centres of the two concrete wythes

It was determined that the shear connectors fail prior to the individual wythes reaching their ultimate moment capacity. The concrete wythes typically reached their cracking moment or slightly beyond their cracking moment when the shear connectors reach their shear capacity. As a result M_1 and M_2 are limited to M_{cr}. Design and Analysis examples using the above equation are presented in Appendix A through Appendix C.

The style of shear connector primarily focused upon throughout this thesis was a galvanized steel 3/4" - #13 standard expanded metal connector. The connectors fit neatly in the joints between the sheets of insulation. The standard expanded metal shear connectors also exhibit behaviour desirable for use in partially composite sandwich panels. The expanded metal connectors are initially stiff and reach their ultimate capacity with little permanent deformation. They yield, and maintain this capacity through a large enough shear deformation that the overall panel behaviour is ductile.

The test panels containing the galvanized steel 3/4" - #13 standard expanded metal shear connectors provided a ductile panel failure. All panels were tested to a mid-span deflection of approximately L/50. The test panels maintained their flexural capacity through the duration of the tests. It was concluded that sandwich panels using fewer prestressing strands and fewer shear connectors than traditional panels can successfully be designed to resist the required design loads.

The recommendations for future research include:

- Perform further research to try and harmonize the design procedures for partially composite members by CSA S16.1-94 with those proposed in this thesis.
- Further research should be performed on the effects of cyclic loading on the connectors. Load cycles could result from wind or thermal gradient fluctuations.
- Research should be done on the influence of connection details that prevent end slip at the base of the panel.
- Extend research to load bearing panels. With partially composite panels the effective EI for buckling computations warrants further study.

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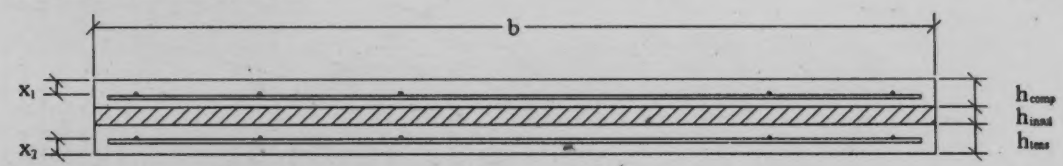
Appendix A (Predicted Behaviour Example)

M – ϕ Diagrams

Develop M – ϕ diagrams for compression and tension wythes.

PANEL INFORMATION



Specimen:	Flexural Test Panel A1
Weight:	32.20 kN
L (length):	3048 mm
b:	2385 mm
h_{comp}:	76 mm
h_{insul}:	76 mm
h_{tens}:	76 mm (ribs are ignored)
x_1:	38 mm
x_2:	26 mm

CONCRETE PROPERTIES

Compression Wythe		Tension Wythe	
ϕ_c:	1.0 (for analysis)	ϕ_c:	1.0 (for analysis)
f_c':	54.8 MPa	f_c':	48.3 MPa
$E_c=4000(f_c')^{1/2}$:	29611 MPa	$E_c=4000(f_c')^{1/2}$:	27799 MPa

Assume Todeschini Concrete Stress – Strain parameters (MacGregor, 2000)

$f_{c(max)} = 0.9\ f_c'$:	49.3 MPa	$f_{c(max)} = 0.9\ f_c'$:	43.5 MPa
$\varepsilon_0 = 1.7\ (f_c'/E_c)$:	0.0031461	$\varepsilon_0 = 1.7\ (f_c'/E_c)$:	0.0029537

STRAND PROPERTIES

Compression Wythe		Tension Wythe	
ϕ_P:	1.0 (for analysis)	ϕ_P:	1.0 (for analysis)
f_{pu}:	1860 MPa	f_{pu}:	1860 MPa
k_p:	0.28	k_p:	0.28
E_p (assumed):	190,000 MPa	E_p (assumed):	190,000 MPa
stress after losses:	0.489 f_{pu}	stress after losses:	0.582 f_{pu}
f_{pe}=0.489f_{pu}:	910 MPa	f_{pe}=0.582f_{pu}:	1083 MPa
ε_{pe}=f_{pe}/E_p:	0.0047871	ε_{pe}=f_{pe}/E_p:	0.0056975
A_{strand}:	54.8 mm^2	A_{strand}:	54.8 mm^2
# of strands:	5	# of strands:	5
A_p:	274 mm^2	A_p:	274 mm^2
d_p:	38 mm	d_p:	50 mm

M – ϕ DIAGRAM (COMPRESSION WYTHE)

Note: The compressive force resulting from shear transfer through the shear connectors between the two wythes is not considered when creating the M – ϕ diagram for the compression wythe. The increased capacity that results from the compression force in the compression wythe is approximately equal to the decrease in capacity experienced by the tension wythe when incorporating the tensile force.

UNCRACKED TRANSFORMED SECTION PROPERTIES

$$n = \frac{E_p}{E_{c(comp)}} = 6.42$$

$$A_g = b \times h_{comp} + (n-1) \times A_{p(comp)} = 1.828 \times 10^5 \, mm^2$$

$$y_t = \frac{\sum (A \times \tilde{y})}{\sum A} = \frac{2385 \times 76 \times 38 + (6.42-1) \times (274 \times 38)}{2385 \times 76 + (6.42-1) \times 274} = 38 mm$$

from extreme tension fibre

$$I = \sum \left(\frac{b \times h^3}{12} + A \times d^2 \right) = 8.725 \times 10^7 \, mm^4$$

DETERMINE ε_{ce}

$$e = h_{comp} - d_p - y_t = 0 mm$$

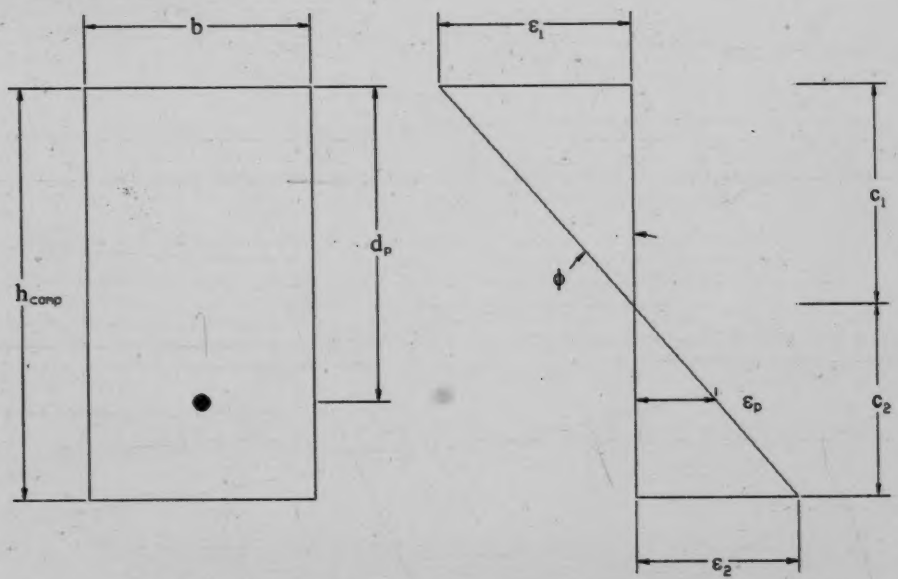
$$f_{ce} = P_e \left(\frac{1}{A_g} + \frac{e^2}{I} \right) = 910 \times 274 \times \left(\frac{1}{1.828 \times 10^5} + \frac{0^2}{8.725 \times 10^7} \right) = 1.37 MPa$$

$$\varepsilon_{ce} = \frac{f_{ce}}{E_c} = \frac{1.37}{29611} = 0.0000462$$

CHOOSE A CURVATURE AND SOLVE FOR THE MOMENT

Choose $\phi = 0.0000051$

Iteration 1



try ε_1 (strain in concrete at extreme compression fibre) = 0.0002000

$$\varepsilon_p = \varepsilon_1 - \tan\phi \times d_p = 0.0002000 - \tan(0.0000051) \times 38 = 0.0000062$$

$$\varepsilon_p - \varepsilon_{pe} - \varepsilon_{ce} = 0.0000062 - 0.0047871 - 0.0000462 = -0.0048271$$

ε_2 (strain in concrete at extreme tension fibre)

$$\varepsilon_2 = \varepsilon_1 - \tan\phi \times h_{comp} = 0.0002000 - \tan(0.0000051) \times 76$$

$$= -0.0001876$$

c_1 (distance from 'compression face' to neutral axis)

if $\varepsilon_1 < 0$ then $c_1 = 0$

if $\varepsilon_1 > 0$ then $c_1 = \dfrac{\varepsilon_1}{\tan\phi}$

therefore $c_1 = 39.2$ mm

c_2 (distance from 'tension face' to neutral axis)

if $\varepsilon_2 < 0$ then $c_2 = 0$

if $\varepsilon_2 > 0$ then $c_2 = \dfrac{\varepsilon_2}{\tan\phi}$

therefore $c_2 = 0.0$ mm

d_t (depth of concrete capable of carrying tension)

if $c_1 > h_{comp}$ then $d_t = 0$

if $c_1 < h_{comp}$ then

$$\text{if } c_1 + 0.6\sqrt{f_c'} \times \frac{c_1}{E_c \varepsilon_1} > h_{comp} \text{ then}$$

$$d_t = h_{comp} - c_1$$

$$\text{if } c_1 + 0.6\sqrt{f_c'} \times \frac{c_1}{E_c \varepsilon_1} < h_{comp} \text{ then}$$

$$d_t = 0.6\sqrt{f_c'} \times \frac{c_1}{E_c \varepsilon_1}$$

therefore $d_t = 29.41$ mm

$f_{c1(avg)}$ (average compressive stress between 'compression face' and neutral axis)

if $\varepsilon_1 < 0$ then $f_{c1(avg)} = 0$

$$\text{if } \varepsilon_1 > 0 \text{ then } f_{c1(avg)} = f_{c(max)} \times \frac{\ln\left(1 + \left(\varepsilon_1 / \varepsilon_0\right)^2\right)}{\left(\varepsilon_1 / \varepsilon_0\right)}$$

therefore $f_{c1(avg)} = 3.13$ MPa

$f_{c2(avg)}$ (average compressive stress between 'tension face' and neutral axis)

if $\varepsilon_2 < 0$ then $f_{c2(avg)} = 0$

$$\text{if } \varepsilon_2 > 0 \text{ then } f_{c2(avg)} = f_{c(max)} \times \frac{\ln\left(1 + \left(\varepsilon_2 / \varepsilon_0\right)^2\right)}{\left(\varepsilon_2 / \varepsilon_0\right)}$$

therefore $f_{c2(avg)} = 0.00$ MPa

$$P_{c1} = f_{c1(avg)} \times b \times c_1 \times 10^{-3} = 3.13 \times 2385 \times 39.2 \times 10^{-3} = 292.6kN$$

$$P_{c2} = f_{c2(avg)} \times b \times c_2 \times 10^{-3} = 0.00 \times 2385 \times 0.0 \times 10^{-3} = 0.0kN$$

f_{cp} (compressive stress at depth of prestressing strand)

if $\varepsilon_p < 0$ then $f_{cp} = 0$

$$\text{if } \varepsilon_p > 0 \text{ then } f_{cp} = \frac{2 \times f_{c(max)} \left(\varepsilon_p / \varepsilon_o\right)}{1 + \left(\varepsilon_p / \varepsilon_o\right)^2}$$

therefore $f_{cp} = 0.19$ MPa

P_p (force carried by the prestressing strand)

if $\left|\left(\varepsilon_p - \varepsilon_{pe} - \varepsilon_{ce}\right)\right| < 0.008$ then

$$P_p = -A_p \times \left(E_p \times \left|\left(\varepsilon_p - \varepsilon_{pe} - \varepsilon_{ce}\right)\right| - f_{cp}\right) \times 10^{-3}$$

if $\left|\left(\varepsilon_p - \varepsilon_{pe} - \varepsilon_{ce}\right)\right| > 0.008$ then

$$\text{if } 1848 - \frac{0.517}{\left|\left(\varepsilon_p - \varepsilon_{pe} - \varepsilon_{ce}\right)\right| - 0.0065} - f_{cp} > 0.98 f_{pu} \text{ then}$$

$$P_p = -A_p \times \left(0.98 f_{pu}\right) \times 10^{-3}$$

$$\text{if } 1848 - \frac{0.517}{\left|\left(\varepsilon_p - \varepsilon_{pe} - \varepsilon_{ce}\right)\right| - 0.0065} - f_{cp} < 0.98 f_{pu} \text{ then}$$

$$P_p = -A_p \times \left(1848 - \frac{0.517}{\left|\left(\varepsilon_p - \varepsilon_{pe} - \varepsilon_{ce}\right)\right| - 0.0065} - f_{cp}\right) \times 10^{-3}$$

therefore Pp = -252.2 kN

P_t (tensile force carried by the concrete)

$$\text{if } \left|\varepsilon_2\right| > 0.6 \times \frac{\sqrt{f_c'}}{E_c} \text{ then}$$

$$P_t = -\tfrac{1}{2} \times 0.6\sqrt{f_c'} \times d_t \times b \times 10^{-3}$$

$$\text{if } \left|\varepsilon_2\right| < 0.6 \times \frac{\sqrt{f_c'}}{E_c} \text{ then}$$

$$P_t = -\tfrac{1}{2} \times E_c \times \left|\varepsilon_2\right| \times d_t \times b \times 10^{-3}$$

therefore P_t = -155.8 kN

e_1 (distance between centroid of uncracked section (at $h_{comp}/2$) and the centre of gravity of the stress diagram between $\varepsilon = 0$ and $\varepsilon = \varepsilon_1$)

if $\varepsilon_1 < 0$ then

$e_1 = 0$

if $\varepsilon_1 > 0$ then

$$e_1 = {}^{h_{comp}}\!/_{2} - \left(1 - \frac{2 \times \left(\left({}^{\varepsilon_1}\!/_{\varepsilon_0} \right) - \tan^{-1}\left({}^{\varepsilon_1}\!/_{\varepsilon_0} \right) \right)}{\left({}^{\varepsilon_1}\!/_{\varepsilon_0} \right)^2 \times \left(\frac{\ln\left(1 + \left({}^{\varepsilon_1}\!/_{\varepsilon_0} \right)^2 \right)}{\left({}^{\varepsilon_1}\!/_{\varepsilon_0} \right)} \right)} \right) \times c_1$$

therefore $e_1 = 24.92$ mm

e_2 (distance between centroid of uncracked section (at $h_{comp}/2$) and the centre of gravity of the stress diagram between $\varepsilon = 0$ and $\varepsilon = \varepsilon_2$)

if $\varepsilon_2 < 0$ then

$e_2 = 0$

if $\varepsilon_2 > 0$ then

$$e_2 = {}^{h_{comp}}\!/_{2} - \left(1 - \frac{2 \times \left(\left({}^{\varepsilon_2}\!/_{\varepsilon_0} \right) - \tan^{-1}\left({}^{\varepsilon_2}\!/_{\varepsilon_0} \right) \right)}{\left({}^{\varepsilon_2}\!/_{\varepsilon_0} \right)^2 \times \left(\frac{\ln\left(1 + \left({}^{\varepsilon_2}\!/_{\varepsilon_0} \right)^2 \right)}{\left({}^{\varepsilon_2}\!/_{\varepsilon_0} \right)} \right)} \right) \times c_2$$

therefore $e_2 = 0.00$ mm

Calculate the moments

$$M_{c1} = P_{c1} \times e_1 \times 10^{-3} = 292.6 \times 24.92 \times 10^{-3} = 7.29 kN \cdot m$$

$$M_{c2} = P_{c2} \times e_2 \times 10^{-3} = 0.0 \times 0.00 \times 10^{-3} = 0.00 kN \cdot m$$

$$M_p = P_p \times \left({}^{h_{comp}}\!/_{2} - d_p \right) \times 10^{-3} = -252.2 \times \left({}^{76}\!/_{2} - 38 \right) \times 10^{-3} = 0.00 kN \cdot m$$

$$M_t = P_t \times \left({}^{h_{comp}}\!/_{2} - c_1 - {}^{2}\!/_{3} d_t \right) \times 10^{-3}$$

$$= -155.8 \times \left({}^{76}\!/_{2} - 39.2 - {}^{2}\!/_{3} \times 29.41 \right) \times 10^{-3} = 3.24 kN \cdot m$$

Calculate M_{sum} and P_{sum}

$$M_{sum} = M_{c1} + M_{c2} + M_p + M_t = 10.54 kN \cdot m$$

$$P_{sum} = P_{c1} + P_{c2} + P_p + P_t = -115.4 kN \neq 0.0 kN$$

Since P_{sum} does not equal 0.0 try a new value for ε_1

Iteration 2

try ε_1 (strain in concrete at extreme compression fibre) = 0.0002500

$$M_{sum} = M_{c1} + M_{c2} + M_p + M_t = 13.69 kN \cdot m$$

$$P_{sum} = P_{c1} + P_{c2} + P_p + P_t = 75.8 kN \neq 0.0 kN$$

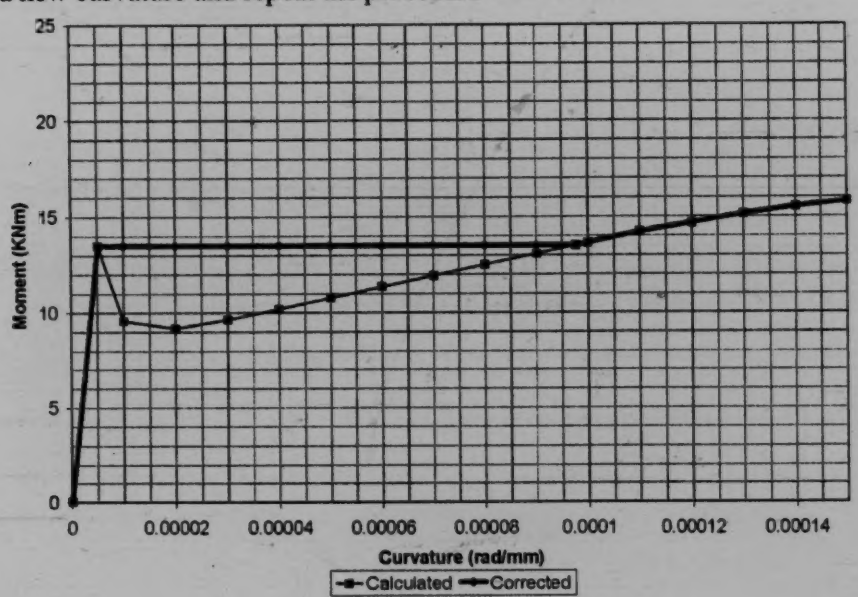
Iteration 3

try ε_1 (strain in concrete at extreme compression fibre) = 0.0002358

$$M_{sum} = M_{c1} + M_{c2} + M_p + M_t = 13.51 kN \cdot m$$

$$P_{sum} = P_{c1} + P_{c2} + P_p + P_t = 0.0 kN \therefore OK$$

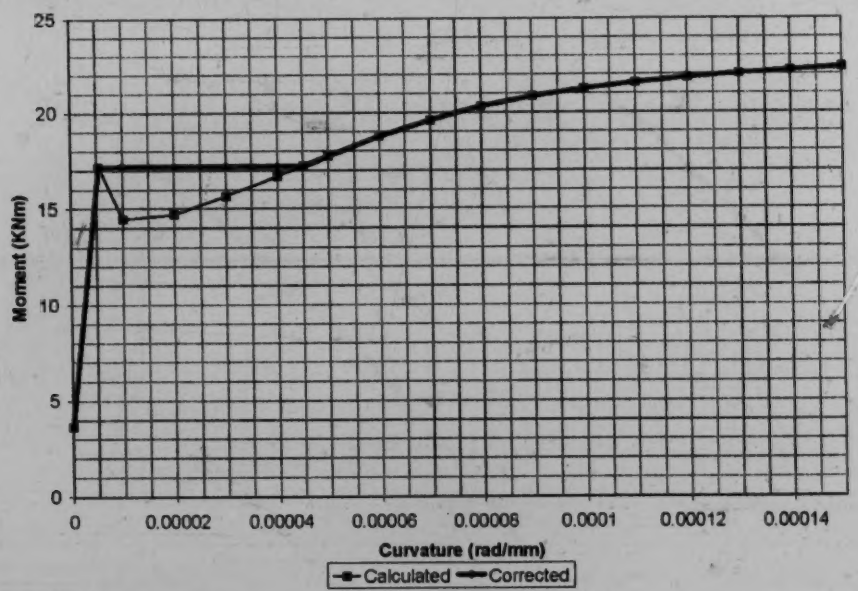
Choose a new curvature and repeat the procedure



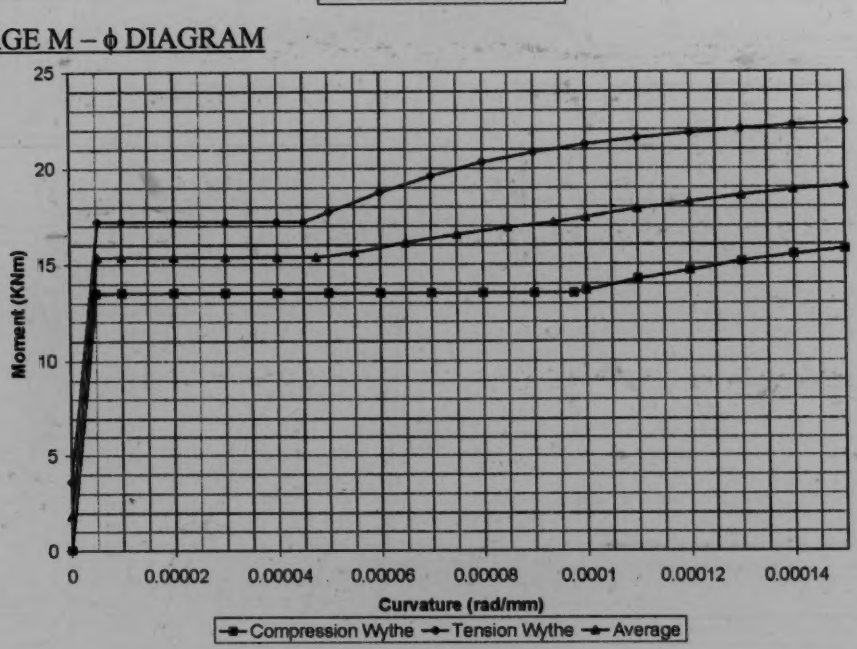
M – ϕ DIAGRAM (TENSION WYTHE)

Same calculations as used for the compression wythe

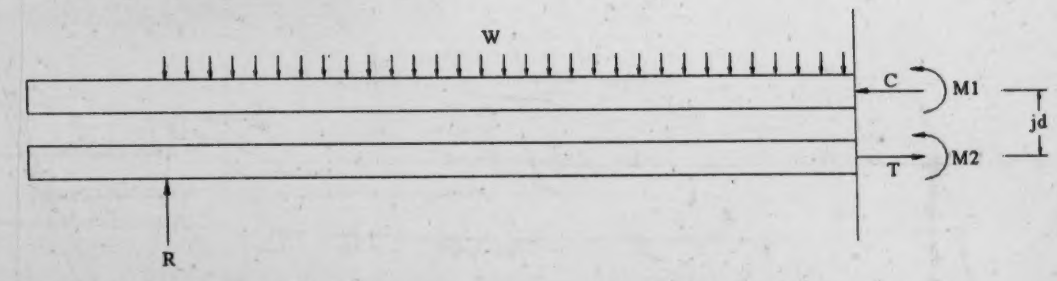
Note: The tensile force resulting from shear transfer through the shear connectors between the two wythes is not considered when creating the M – ϕ diagram for the tension wythe. The decreased capacity that results from the tension force in the tension wythe is approximately equal to the increase in capacity experienced by the compression wythe when incorporating the compressive force.



AVERAGE M – ϕ DIAGRAM



Load – Deflection Diagram



$M_n = M_1 + M_2 + V \times jd$

where V is the lesser of (V_{sc} + V_{insul}) or ($A_p f_{pr}$ for tension wythe)

Choose a curvature (ϕ = 0.00000525)

DETERMINE M_1 AND M_2 FROM THE M – ϕ DIAGRAMS

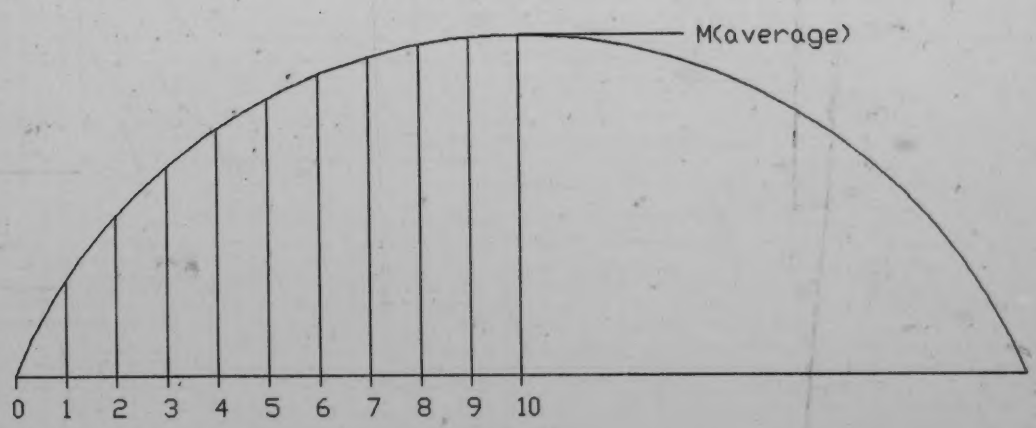
M_1 (Bending Moment of the compression wythe) = 13.51 kN·m

M_2 (Bending Moment of the tension wythe) = 17.21 kN·m

DETERMINE MID-SPAN DEFLECTION USING MOMENT – AREA THEORM

M_{avg} = 15.36 kN·m

$M = \frac{wL^2}{8}$ $\quad 15.36 = \frac{w \times 3.048^2}{8}$ therefore w = 13.23 kN/m

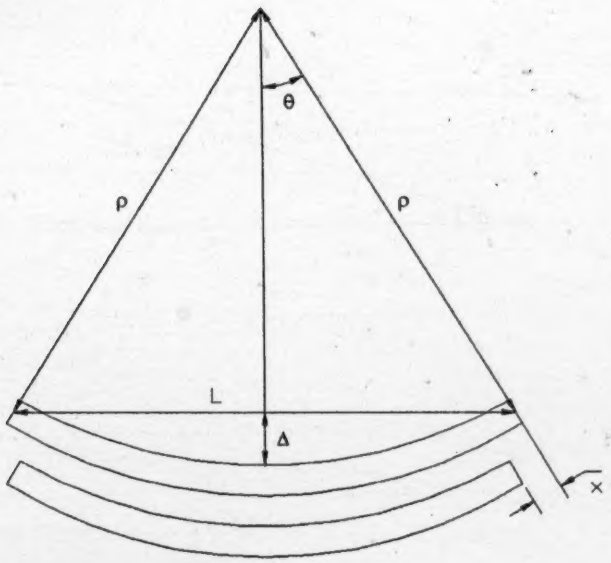


Position	Distance from end (mm)	Moment (kN·m)	M/EI (curvature) from M-ϕ Diagram	Area under M/EI Diagram	A·δ
0	0.0	0.000	0.00000000		
				0.0000328	0.003
1	152.4	2.918	0.00000043		
				0.0001426	0.035
2	304.8	5.530	0.00000144		
				0.0002877	0.111
3	457.2	7.834	0.00000233		
				0.0004147	0.223
4	609.6	9.830	0.00000311		
				0.0005235	0.360
5	762.0	11.520	0.00000376		
				0.0006142	0.516
6	914.4	12.902	0.00000430		
				0.0006867	0.681
7	1066.8	13.978	0.00000471		
				0.0007412	0.848
8	1219.2	14.746	0.00000501		
				0.0007774	1.007
9	1371.6	15.206	0.00000519		
				0.0007956	1.152
10	1524.0	15.360	0.00000525		
			Σ	0.0050163	4.936

$$\delta = \frac{\sum(A \cdot \delta)}{\sum A} = \frac{4.936}{0.0050163} = 984.033mm$$

$$\Delta = \sum A \times \delta = 0.0050163 \times 984.033 = 4.936mm$$

ESTIMATE THE END SLIP



Assume $\Delta = \frac{5wL^4}{384EI}$ and $\phi = \frac{M}{EI} = \frac{wL^2}{8EI}$

therefore $\phi = \frac{48\Delta}{5L^2} = \frac{48 \times 4.936}{5 \times 3048^2} = 5.101 \times 10^{-6}$

$$\rho = 1/\phi = 1/5.101 \times 10^{-6} = 196.05 \times 10^3 \, mm$$

$$\theta = \sin^{-1}\left(\frac{L/2}{\rho}\right) = \sin^{-1}\left(\frac{3048/2}{196.05 \times 10^{-3}}\right) = 0.445^o$$

$$Slip = x \approx \tan\theta \times h_{insul} = \tan(0.445) \times 76 = 0.591 mm$$

DETERMINE V·jd

# Shear Connectors	Distance from midspan (mm)	Estimated Slip (mm)	Connector Shear Force Test 1 (kN)	Connector Shear Force Test 2 (kN)	Connector Shear Force Test 3 (kN)	Average Shear Force / connector (kN)
4	676.5	$\frac{676.5}{(3048/2)} \times 0.591$ = 0.262 mm	14.10	12.73	N/A	13.41

$V = 4 \times 13.41 = 53.65kN$

$jd = {}^{h_{comp}}/_{2} + h_{insul} + {}^{h_{tens}}/_{2} = {}^{76}/_{2} + 76 + {}^{76}/_{2} = 152.0mm$ (distance between centroids of concrete wythes)

$V \cdot jd = 53.65 \times 152.0 \times 10^{-3} = 8.16kN \cdot m$

MOMENT CAPACITY EXCLUDING INSULATION FRICTION CONTRIBUTION

$M_n = 13.51 + 17.21 + 8.16 = 38.88kN \cdot m$

MOMENT CAPACITY WITH INSULATION FRICTION

Iteration 1

from $M = \frac{wL^2}{8} \Rightarrow 38.88 = \frac{w \times (3048 \times 10^{-3})^2}{8} \Rightarrow w = 33.48 {}^{kN}/_{m}$

$N = {}^{1}/_{2} \times 33.48 {}^{kN}/_{m} \times 3.048m + \frac{{}^{1}/_{2} \times 32.20}{2} = 59.07kN$ (Normal force acting between locations of maximum moment and zero moment (i.e. half of panel))

Note: Because the panel was tested in the horizontal position the weight of the top wythe adds an additional normal force which is included in the above calculation. The shear connectors also result in a clamping force between the two concrete wythes. This clamping force also existed in the unit shear tests and therefore does not need to be included in the above calculation.

$V_{insul} = \mu N = 0.65 \times 59.07 = 38.40kN$

$jd = \frac{h_{comp}}{2} + h_{insul} + \frac{h_{tens}}{2} = \frac{76}{2} + 76 + \frac{76}{2} = 152mm$

$M_{insul} = 38.40 \times 152.0 \times 10^{-3} = 5.84kN \cdot m$

$M_n = 38.88 + 5.84 = 44.72kN \cdot m$

Iteration 2

$$\text{from } M = \frac{wL^2}{8} \Rightarrow 44.72 = \frac{w \times \left(3048 \times 10^{-3}\right)^2}{8} \Rightarrow w = 38.51 \, {}^{kN}/_{m}$$

$$N = {}^{1}/_{2} \times 38.51 \, {}^{kN}/_{m} \times 3.048m + \frac{{}^{1}/_{2} \times 32.20}{2} = 66.73kN$$

$$V_{insul} = \mu N = 0.65 \times 66.73 = 43.38kN$$

$$jd = \frac{h_{comp}}{2} + h_{insul} + \frac{h_{tens}}{2} = \frac{76}{2} + 76 + \frac{76}{2} = 152mm$$

$$M_{insul} = 43.38 \times 152.0 \times 10^{-3} = 6.59kN \cdot m$$

$$M_n = 38.88 + 6.76 = 45.47kN \cdot m$$

Iteration 3

$$\text{from } M = \frac{wL^2}{8} \Rightarrow 45.47 = \frac{w \times \left(3048 \times 10^{-3}\right)^2}{8} \Rightarrow w = 39.16 \, {}^{kN}/_{m}$$

$$N = {}^{1}/_{2} \times 39.16 \, {}^{kN}/_{m} \times 3.048m + \frac{{}^{1}/_{2} \times 32.20}{2} = 67.73kN$$

$$V_{insul} = \mu N = 0.65 \times 67.73 = 44.02kN$$

$$jd = \frac{h_{comp}}{2} + h_{insul} + \frac{h_{tens}}{2} = \frac{76}{2} + 76 + \frac{76}{2} = 152mm$$

$$M_{insul} = 44.02 \times 152.0 \times 10^{-3} = 6.69kN \cdot m$$

$$M_n = 38.88 + 6.69 = 45.57kN \cdot m$$

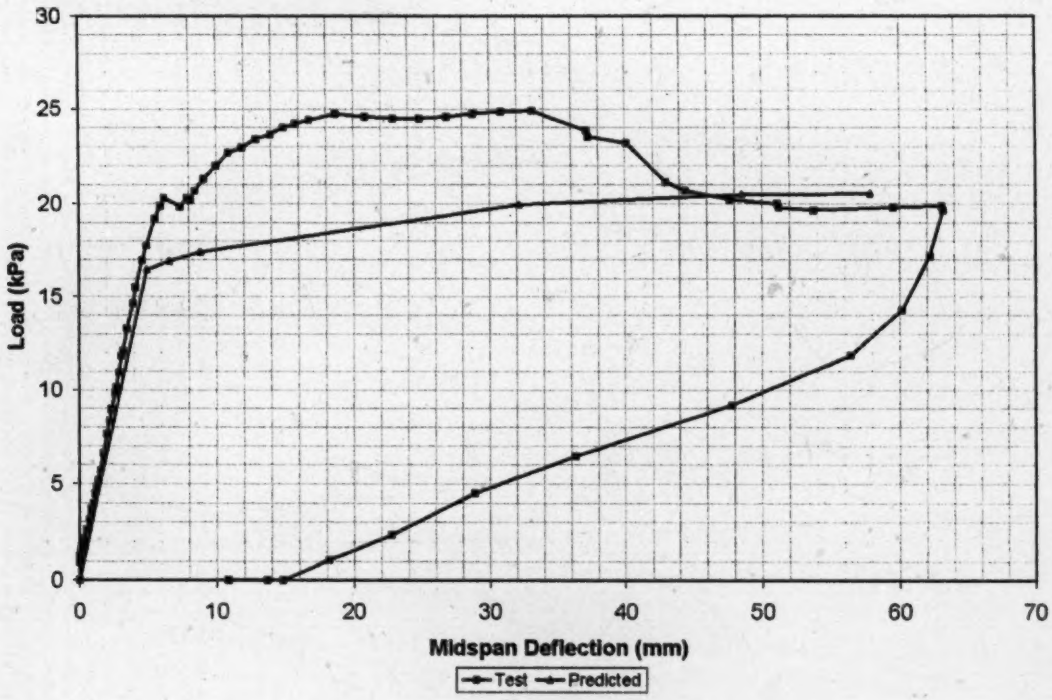
CALCULATED CAPACITY OF WALL PANEL

$$M_n = 45.57kN \cdot m$$

$$w_n = 16.45kPa$$

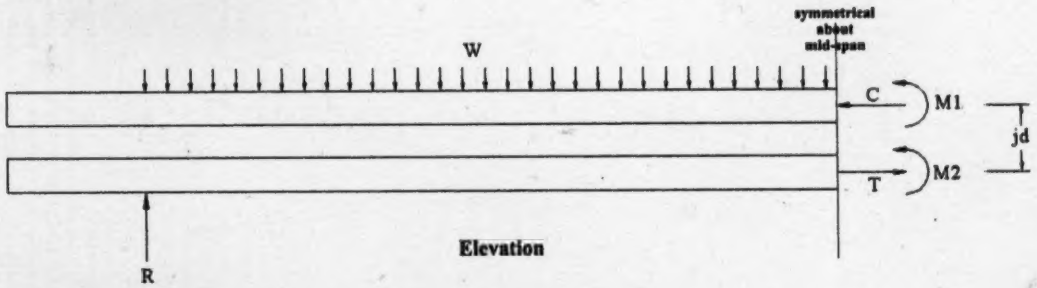
Choose a new curvature and repeat procedure

LOAD – DEFLECTION DIAGRAM



Appendix B (Simplified Ultimate Capacity Example)

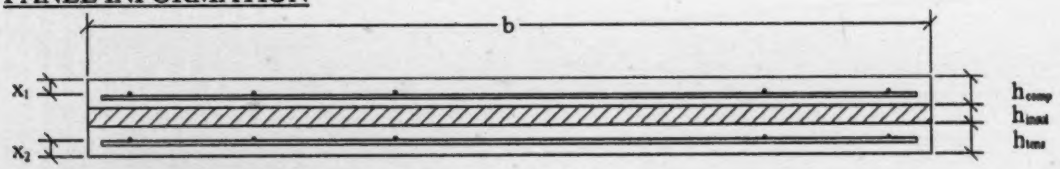
Analysis Calculations



$M_n = M_1 + M_2 + V \times jd$

where V is the lesser of (V_{sc} + V_{insul}) or $A_p f_{pr}$ for tension wythe)
and T = C = V

PANEL INFORMATION



Specimen:	Flexural Test Panel A1
Weight:	32.20 kN
L (Length):	3048 mm
b:	2385 mm
h_{comp}:	76 mm
h_{insul}:	76 mm
h_{tens}:	76 mm (ribs are ignored)
x_1:	38 mm
x_2:	26 mm

CONCRETE PROPERTIES

Compression Wythe		Tension Wythe	
ϕ_c:	1.0 (for analysis)	ϕ_c:	1.0 (for analysis)
f_c':	54.8 MPa	f_c':	48.3 MPa
α_1=0.85-0.0015 f_c':	0.768	α_1=0.85-0.0015 f_c':	0.778
β_1=0.97-0.0025 f_c':	0.833	β_1=0.97-0.0025 f_c':	0.849
E_c=4000$(f_c')^{1/2}$:	29611 MPa	E_c=4000$(f_c')^{1/2}$:	27799 MPa

STRAND PROPERTIES

Compression Wythe		Tension Wythe	
ϕ_P:	1.0 (for analysis)	ϕ_P:	1.0 (for analysis)
f_{pu}:	1860 MPa	f_{pu}:	1860 MPa
k_p:	0.28	k_p:	0.28
E_p (assumed):	190,000 MPa	E_p (assumed):	190,000 MPa
stress after losses:	0.489 f_{pu}	stress after losses:	0.582 f_{pu}
$f_{pe}=0.489f_{pu}$:	910 MPa	$f_{pe}=0.582f_{pu}$:	1083 MPa
$\varepsilon_{pe}=f_{pe}/E_p$:	0.00479	$\varepsilon_{pe}=f_{pe}/E_p$:	0.00570
A_{strand}:	54.8 mm^2	A_{strand}:	54.8 mm^2
# of strands:	5	# of strands:	5
A_p:	274 mm^2	A_p:	274 mm^2
d_p:	38 mm	d_p:	202 mm

UNCRACKED TRANSFORMED SECTION PROPERTIES
(Assuming fully-composite behaviour)

$$n = \frac{E_p}{E_{c(comp)}} = 6.42$$

$$A_g = b\times\left(h_{comp} + h_{tens}\right) + (n-1)\times\left(A_{p(comp)} + A_{p(tens)}\right) = 3.655\times10^5\ mm^2$$

$$y_t = \frac{\sum(A\times\tilde{y})}{\sum A} = \frac{2385\times76\times38 + 2385\times76\times190 + (6.42-1)\times(274\times26 + 274\times190)}{2385\times76 + 2385\times76 + (6.42-1)\times(274+274)}$$

= 113.95 mm from extreme tension fibre

$$I = \sum\left(\frac{b\times h^3}{12} + A\times d^2\right) = 2.272\times10^9\ mm^4$$

SHEAR CONNECTOR PROPERTIES

Type: 3/4"-#13 Galvanized Standard Expanded Metal
ϕ_{sc}: 1.0 (for analysis)
Capacity / Connector: 25.5 kN

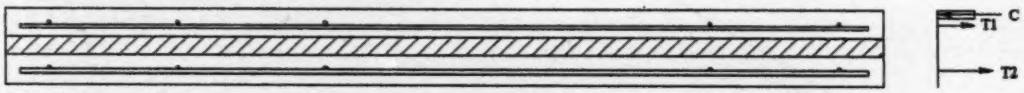
Connector distance from mid-span	# of connectors	Factored capacity of connector	Capacity
676.5 mm	4	25.5 kN	102.1 kN

Factored Capacity: 102.1 kN (capacity of connectors between locations of maximum moment and zero moment (i.e. half of panel))

INSULATION PROPERTIES

Type: DOW Panelmate Insulation
ϕ_{insul}: 1.0 (for analysis)
μ: 0.65 (coefficient of friction between insulation and concrete)
μ_f: 1.0 x 0.65 = 0.65

ASSUME FULLY COMPOSITE PANEL



$C = \phi_c \alpha_1 f_c' ba = 1.0 \times 0.768 \times 54.8 \times 2385 \times (0.833c) \times 10^{-3} = 83.613c$

$T = \phi_p A_p f_{pr}$

as a start, assume $f_{pr} = f_{pu}\left(1 - k_p \, {}^{c}/_{d_p}\right)$

$T_1 = 1.0 \times 274 \times 1860 \times \left(1 - 0.28 \times {}^{c}/_{38}\right) \times 10^{-3} = 509.64(1 - 0.007368c)$

$T_2 = 1.0 \times 274 \times 1860 \times \left(1 - 0.28 \times {}^{c}/_{202}\right) \times 10^{-3} = 509.64(1 - 0.001386c)$

from $C = T_1 + T_2$ solve for c

$c = 11.576mm$

Iteration 1

Compression Wythe:

$$\varepsilon_{p(1)} = \frac{0.0035 \times (d_{p(1)} - c)}{c} = \frac{0.0035 \times (38 - 11.576)}{11.576} = 0.00799$$

$$e_{(1)} = h_{comp} + h_{insul} + h_{tens} - x_1 - y_t = 76 + 76 + 76 - 38 - 113.95 = 76.05mm$$

$$f_{ce(1)} = P_e\left(\frac{1}{A_g} + \frac{e^2}{I}\right) = 910 \times 274 \times \left(\frac{1}{3.655 \times 10^5} + \frac{76.05^2}{2.272 \times 10^9}\right) = 1.32MPa$$

$$\varepsilon_{ce(1)} = \frac{f_{ce(1)}}{E_{c(1)}} = \frac{1.32}{29611} = 0.00005$$

$$\varepsilon_{ps(1)} = \varepsilon_{pe(1)} + \varepsilon_{ce(1)} + \varepsilon_{p(1)} = 0.00479 + 0.00005 + 0.00799 = 0.01283$$

if $\varepsilon_{ps(1)} > 0.008$ then:

$$f_{pr(1)} = 1848 - \frac{0.517}{\varepsilon_{ps(1)} - 0.0065}$$

if $\varepsilon_{ps(1)} < 0.008$ then:

$$f_{pr(1)} = \varepsilon_{ps(1)} \times E_p$$

therefore

$$f_{pr(1)} = 1848 - \frac{0.517}{0.01283 - 0.0065} = 1766.2MPa$$

if $f_{pr(1)} > 0.98\ f_{pu}$ then:

$$f_{pr(1)} = 0.98 f_{pu}$$

if $f_{pr(1)} < 0.98\ f_{pu}$ then:

$$f_{pr(1)} = f_{pr(1)}$$

therefore

$$f_{pr(1)} = 1766.2MPa$$

$$T_1 = \phi_p A_p f_{pr(1)} = 1.0 \times 274 \times 1766.2 \times 10^{-3} = 483.9kN$$

Tension Wythe:

$$\varepsilon_{p(2)} = \frac{0.0035 \times (d_{p(2)} - c)}{c} = \frac{0.0035 \times (202 - 11.576)}{11.576} = 0.05758$$

$$e_{(2)} = y_t - x_2 = 113.95 - 26 = 87.95mm$$

$$f_{ce(2)} = P_e\left(\frac{1}{A_g} + \frac{e^2}{I}\right) = 1083 \times 274 \times \left(\frac{1}{3.655 \times 10^5} + \frac{87.95^2}{2.272 \times 10^9}\right) = 1.82MPa$$

$$\varepsilon_{ce(2)} = \frac{f_{ce2}}{E_{c2}} = \frac{1.82}{27799} = 0.00007$$

$$\varepsilon_{ps(2)} = \varepsilon_{pe(2)} + \varepsilon_{ce(2)} + \varepsilon_{p(2)} = 0.00570 + 0.00007 + 0.05758 = 0.06334$$

if $\varepsilon_{ps(2)} > 0.008$ then:

$$f_{pr(2)} = 1848 - \frac{0.517}{\varepsilon_{ps(2)} - 0.0065}$$

if $\varepsilon_{ps(2)} < 0.008$ then:

$$f_{pr(2)} = \varepsilon_{ps(2)} \times E_p$$

therefore

$$f_{pr(2)} = 1848 - \frac{0.517}{0.06334 - 0.0065} = 1838.9MPa$$

if $f_{pr(2)} > 0.98\ f_{pu}$ then:

$$f_{pr(2)} = 0.98 f_{pu}$$

if $f_{pr(2)} < 0.98\ f_{pu}$ then:

$$f_{pr(2)} = f_{pr(2)}$$

therefore

$$f_{pr(2)} = 0.98 f_{pu} = 1822.8MPa$$

$$T_2 = \phi_p A_p f_{pr(2)} = 1.0 \times 274 \times 1822.8 \times 10^{-3} = 499.4kN$$

from $C = T_1 + T_2$ solve for c

$$c = 11.764mm$$

Iteration 2

perform same calculations as Iteration 1 using $c_{average} = (11.576 + 11.764) / 2 = 11.670mm$

C = 83.613c
T_1 = 483.6 kN
T_2 = 499.4 kN

from $C = T_1 + T_2$ solve for c

$c = 11.760mm$

M_n if panel is fully composite

$$M_n = \left[C \times (c - a/2) + T_1 \times (d_{p(1)} - c) + T_2 \times (d_{p(2)} - c)\right] \times 10^{-3} = 111.62kN \cdot m$$

CALCULATION FOR PARTIALLY COMPOSITE BEHAVIOUR

MOMENT TRANSFERRED THROUGH SHEAR CONNECTORS

$M_{sc} = V_{sc} \times jd$

V_{sc} is lesser of capacity of shear connectors or T_2

102.1 kN << 499.4 kN therefore V_{sc} = 102.1 kN

$$jd = \frac{h_{comp}}{2} + h_{insul} + \frac{h_{tens}}{2} = \frac{76}{2} + 76 + \frac{76}{2} = 152mm$$

$$M_{sc} = 102.1 \times 152.0 \times 10^{-3} = 15.52kN \cdot m$$

DETERMINE M_1 – MOMENT RESISTANCE OF COMPRESSION WYTHE

Uncracked Transformed Section Properties

$$n = \frac{E_p}{E_c} = \frac{190{,}000}{29611} = 6.42$$

$$A_t = b \times h_{comp} + (n-1) \times A_{p(comp)} = 1.827 \times 10^5 mm^2$$

$$y_t = \frac{\sum (A \times \tilde{y})}{\sum A} = \frac{2385 \times 76 \times 38 + (6.42 - 1) \times 274 \times 38}{2385 \times 76 + (6.42 - 1) \times 274} = 38.0mm$$

from extreme tension fibre

$$e = y_t + x_1 - h_{comp} = 38 + 38 - 76 = 0mm$$

$$I = \sum \left(\frac{b \times h^3}{12} + A \times d^2 \right) = 8.725 \times 10^7 mm^4$$

$$S = \frac{I}{y_t} = \frac{8.725 \times 10^7}{38.0} = 2.296 \times 10^6 mm^3$$

Determine M_1

Note: The compressive force resulting from shear transfer through the shear connectors between the two wythes is not considered when calculating M_1. The increased capacity that results from the compression force in the compression wythe is approximately equal to the decrease in capacity experienced by the tension wythe when incorporating the tensile force.

Note: It was seen from the Predicted Behaviour Example (Appendix A) that the concrete wythes do not fully develop their individual plastic moment resistance prior to the shear connectors failing. As a result M_1 is limited to M_{cr} for simplicity.

$$M_{cr} = \left(\frac{P_e}{A_t} + \frac{P_e \times e}{S} + 0.6\lambda\sqrt{f_c'}\right) \times S \times 10^{-6}$$

$$= \left(\frac{910 \times 274}{1.827 \times 10^5} + \frac{910 \times 274 \times 0}{2.296 \times 10^6} + 0.6 \times 1 \times \sqrt{54.8}\right) \times 2.296 = 13.33 kN \cdot m$$

DETERMINE M_2 – MOMENT RESISTANCE OF TENSION WYTHE

Uncracked Transformed Section Properties

$$n = \frac{E_p}{E_c} = \frac{190{,}000}{27799} = 6.83$$

$$A_t = b \times h_{tens} + (n-1) \times A_{p(tens)} = 1.829 \times 10^5 \, mm^2$$

$$y_t = \frac{\sum(A \times \tilde{y})}{\sum A} = \frac{2385 \times 76 \times 38 + (6.83-1) \times 274 \times 26}{2385 \times 76 + (6.83-1) \times 274} = 37.9 mm$$

from extreme tension fibre

$$e = y_t - x_2 = 37.9 - 26 = 11.9 mm$$

$$I = \sum\left(\frac{b \times h^3}{12} + A \times d^2\right) = 8.747 \times 10^7 \, mm^4$$

$$S = \frac{I}{y_t} = \frac{8.747 \times 10^7}{37.9} = 2.308 \times 10^6 \, mm^3$$

Determine M_2

Note: The tensile force resulting from shear transfer through the shear connectors between the two wythes is not considered when calculating M_2. The decreased capacity that results from the tension force in the tension wythe is approximately equal to the increase in capacity experienced by the compression wythe when incorporating the compression force.

Note: It was seen from the Predicted Behaviour Example (Appendix A) that the concrete wythes do not fully develop their individual plastic moment resistance prior to the shear connectors failing. As a result M_2 is limited to M_{cr} for simplicity.

$$M_{cr} = \left(\frac{P_e}{A_t} + \frac{P_e \times e}{S} + 0.6\lambda\sqrt{f_c'}\right) \times S \times 10^{-6}$$

$$= \left(\frac{1083 \times 274}{1.829 \times 10^5} + \frac{1083 \times 274 \times 11.9}{2.308 \times 10^6} + 0.6 \times 1 \times \sqrt{48.3}\right) \times 2.308 = 16.90 kN \cdot m$$

ULTIMATE BENDING MOMENT CAPACITY EXCLUDING CONTRIBUTION FROM INSULATION FRICTION

$$M_n = 13.33 + 16.90 + 15.52 = 45.75 kN \cdot m$$

MOMENT CAPACITY WITH INSULATION FRICTION

Iteration 1

$$\text{from } M = \frac{wL^2}{8} \Rightarrow 45.75 = \frac{w \times (3048 \times 10^{-3})^2}{8} \Rightarrow w = 39.40 \, {}^{kN}/_{m}$$

$$N = {}^{1}/_{2} \times 39.40 \, {}^{kN}/_{m} \times 3.048 m + \frac{{}^{1}/_{2} \times 32.20}{2} = 68.10 kN$$ (Normal force acting between locations of maximum moment and zero moment (i.e. half of panel))

Note: Because the panel was tested in the horizontal position the weight of the top wythe adds an additional normal force which is included in the above calculation. The shear connectors also result in a clamping force between the two concrete wythes. This clamping force also existed in the unit shear tests and therefore does not need to be included in the above calculation.

$$V_{insul} = \mu N = 0.65 \times 68.10 = 44.26 kN$$

$$jd = \frac{h_{comp}}{2} + h_{insul} + \frac{h_{tens}}{2} = \frac{76}{2} + 76 + \frac{76}{2} = 152 mm$$

$$M_{insul} = 44.26 \times 152.0 \times 10^{-3} = 6.73 kN \cdot m$$

$$M_n = 45.75 + 6.73 = 52.48 kN \cdot m$$

Iteration 2

$$\text{from } M = \frac{wL^2}{8} \Rightarrow 52.48 = \frac{w \times (3048 \times 10^{-3})^2}{8} \Rightarrow w = 45.19 \, {}^{kN}/_{m}$$

$$N = {}^{1}/_{2} \times 45.19 \, {}^{kN}/_{m} \times 3.048 m + \frac{{}^{1}/_{2} \times 32.20}{2} = 76.92 kN$$

$$V_{insul} = \mu N = 0.65 \times 76.92 = 50.00 kN$$

$$jd = \frac{h_{comp}}{2} + h_{insul} + \frac{h_{tens}}{2} = \frac{76}{2} + 76 + \frac{76}{2} = 152 mm$$

$$M_{insul} = 50.00 \times 152.0 \times 10^{-3} = 7.60 kN \cdot m$$

$$M_n = 45.75 + 7.60 = 53.35 kN \cdot m$$

Iteration 3

$$\text{from } M = \frac{wL^2}{8} \Rightarrow 53.35 = \frac{w \times \left(3048 \times 10^{-3}\right)^2}{8} \Rightarrow w = 45.94 \, {}^{kN}/_{m}$$

$$N = {}^{1}/_{2} \times 45.94 \, {}^{kN}/_{m} \times 3.048m + \frac{{}^{1}/_{2} \times 32.20}{2} = 78.06kN$$

$$V_{insul} = \mu N = 0.65 \times 78.06 = 50.74kN$$

$$jd = \frac{h_{comp}}{2} + h_{insul} + \frac{h_{tens}}{2} = \frac{76}{2} + 76 + \frac{76}{2} = 152mm$$

$$M_{insul} = 50.74 \times 152.0 \times 10^{-3} = 7.71kN \cdot m$$

$$M_n = 45.75 + 7.71 = 53.46kN \cdot m$$

CALCULATED CAPACITY OF PARTIALLY COMPOSITE WALL PANEL

$$M_n = 53.46kN \cdot m$$

$$w_n = 19.30kPa$$

TEST / PREDICTED RATIO

$$w_{test} = 24.90KPa$$

$$M_{test} = \frac{wL^2}{8} = \frac{(24.90 \times 2.385) \times 3.048^2}{8} = 68.96KN \cdot m$$

$$\frac{M_{test}}{M_{predicted}} = \frac{68.96}{53.46} \times 100\% = 129.0\%$$

% COMPOSITE ACTION

Eqn. [2.4]

$$S_{specimen} = \frac{w}{\Delta} = \frac{(19.89kPa \times 2.385m)}{5.93mm} = 8.000(N/mm)/mm$$

$$\Delta = \frac{5wL^4}{384EI}$$

$$S_{non-composite} = \frac{w}{\Delta} = \frac{384EI}{5L^4} = \frac{384 \times 28705 \times 0.1745 \times 10^9}{5 \times 3048^4} = 4.457(N/mm)/mm$$

$$S_{composite} = \frac{w}{\Delta} = \frac{384EI}{5L^4} = \frac{384 \times 28705 \times 2.2721 \times 10^9}{5 \times 3048^4} = 58.035(N/mm)/mm$$

$$\%Composite = \left(\frac{8.000 - 4.457}{58.035 - 4.457}\right) \times 100\% = 6.6\%$$

Eqn. [4.2]

Excluding insulation friction

$$\%Composite = \frac{V_{sc}}{T_2} \times 100\% \text{ but not greater than } 100\%$$

$$\%Composite = \frac{102.1}{499.4} \times 100\% = 20.4\%$$

Including insulation friction

$$\%Composite = \frac{V_{sc} + V_{insul}}{T_2} \times 100\% \text{ but not greater than } 100\%$$

$$\%Composite = \frac{102.1 + 50.7}{499.4} \times 100\% = 30.6\%$$

Appendix C (Design Example)

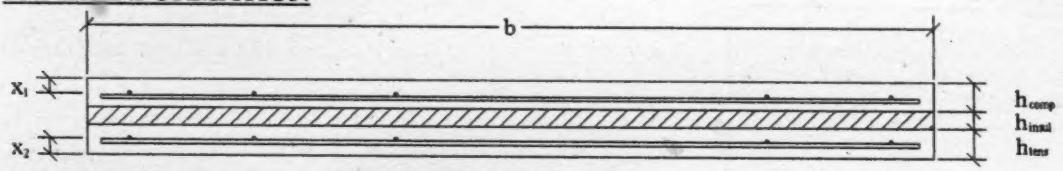
Design Calculations

Design a 9754 mm tall panel with a 610 mm parapet for a service wind load of 1.0 kPA. Include handling and erection stress checks.

Design Codes: NBCC'95
CSA A23.3-94

Design Loads: 1.0 kPa service wind load
Handling and erection loads

PANEL INFORMATION



Specimen:	Flexural Test Panel B1
Total length of panel:	9754 mm
L (Length between supports):	9144 mm
b:	2385 mm
h_{comp}:	76.2 mm
h_{insul}:	101.6 mm
h_{tens}:	76.2 mm
x_1:	38.1 mm
x_2:	38.1 mm

CONCRETE PROPERTIES

Compression Wythe		Tension Wythe	
ϕ_c:	0.65 (for design)	ϕ_c:	0.65 (for design)
Initial concrete properties at time of stripping			
f_{ci}':	20.0 MPa	f_{ci}':	20.0 MPa
$\alpha_{1i}=0.85-0.0015\ f_{ci}'$:	0.820	$\alpha_{1i}=0.85-0.0015\ f_{ci}'$:	0.820
$\beta_{1i}=0.97-0.0025\ f_{ci}'$:	0.920	$\beta_{1i}=0.97-0.0025\ f_{ci}'$:	0.920
$E_{ci}=4000(f_{ci}')^{1/2}$:	17889 MPa	$E_{ci}=4000(f_{ci}')^{1/2}$:	17889 MPa
28-day concrete properties			
f_c':	35.0 MPa	f_c':	35.0 MPa
$\alpha_1=0.85-0.0015\ f_c'$:	0.798	$\alpha_1=0.85-0.0015\ f_c'$:	0.798
$\beta_1=0.97-0.0025\ f_c'$:	0.883	$\beta_1=0.97-0.0025\ f_c'$:	0.883
$E_c=4000(f_c')^{1/2}$:	23664 MPa	$E_c=4000(f_c')^{1/2}$:	23664 MPa

STRAND PROPERTIES

Compression Wythe		Tension Wythe	
ϕ_P:	0.9 (for design)	ϕ_P:	0.9 (for design)
f_{pu}:	1860 MPa	f_{pu}:	1860 MPa
k_p:	0.28	k_p:	0.28
E_p:	190,000 MPa	E_p:	190,000 MPa
stressed to:	0.70 f_{pu}	stressed to:	0.70 f_{pu}
after transfer:	0.68 f_{pu}	after transfer:	0.68f_{pu}
after all losses:	0.60 f_{pu} (assumed)	after all losses:	0.60 f_{pu} (assumed)
f_{pe}=0.60f_{pu}:	1116 MPa	f_{pe}=0.60f_{pu}:	1116 MPa
$\varepsilon_{pe}=f_{pe}/E_p$:	0.00587	$\varepsilon_{pe}=f_{pe}/E_p$:	0.00587
A_{strand}:	54.8 mm^2	A_{strand}:	54.8 mm^2
# of strands:	4	# of strands:	4
A_p:	219.2 mm^2	A_p:	219.2 mm^2
d_p:	38.1 mm	d_p:	215.9 mm

Note: According to CPCI 3.11.2 the prestressed reinforcement should provide an average prestress after all losses of not less than 1.0 MPa

MPa = 1116 MPa x (54.8 mm^2 x # of strands) / (2385 mm x 76.2 mm)
Therefore 3 strand would be adequate (try 4 strands per wythe)

To avoid eccentric prestressing each wythe should be prestressed concentrically.

GROSS UNCRACKED NON-COMPOSITE SECTION PROPERTIES

$$A_{g(comp)} = A_{g(tens)} = b \times h = 1.817 \times 10^5 \, mm^2$$

$$y_{t(comp)} = y_{t(tens)} = \frac{\sum (A \times \tilde{y})}{\sum A} = 38.1 mm \text{ from extreme tension fibre}$$

$$I_{comp} = I_{tens} = \frac{b \times h^3}{12} = 0.088 \times 10^9 \, mm^4 \text{ therefore}$$

$$I_g = I_{comp} + I_{tens} = 0.176 \times 10^9 \, mm^4 \text{ (or } 0.074 \text{ x } 10^9 \text{ mm}^4\text{)}$$

$$S_{comp} = S_{tens} = \frac{I}{y_t} = \frac{0.088 \times 10^9}{38.1} = 2.308 \times 10^6 \, mm^3$$

GROSS UNCRACKED COMPOSITE SECTION PROPERTIES

$$A_g = b \times \left(h_{comp} + h_{tens}\right) = 3.635 \times 10^5 \, mm^2 \text{ (or 152,400 mm}^2\text{/m)}$$

$$y_t = \frac{\sum\left(A \times \tilde{y}\right)}{\sum A} = 127.0mm \text{ from extreme tension fibre}$$

$$I = \sum\left(\frac{b \times h^3}{12} + A \times d^2\right) = 3.048 \times 10^9 \, mm^4 \text{ (or 1.278 x 10}^9 \text{ mm}^4\text{/m)}$$

$$S_{comp} = S_{tens} = \frac{I}{y_t} = \frac{3.048 \times 10^9}{127.0} = 24.004 \times 10^6 \, mm^3 \text{ (or 10.064 x 10}^6 \text{ mm}^3\text{/m)}$$

REINFORCING STEEL PROPERTIES

ϕ_s: 0.85
f_y: 400 MPa

SHEAR CONNECTOR PROPERTIES

Type: 3/4"-#13 Galvanized Standard Expanded Metal
ϕ_{sc}: 0.65
Capacity / Connector: 24.9 kN (Factored Capacity = 16.19 kN)

INSULATION

Type: DOW Panelmate Insulation

FACTORED DESIGN LOADS

Wind:

$w_f = 1.5\ (1.0) = 1.5$ kPa

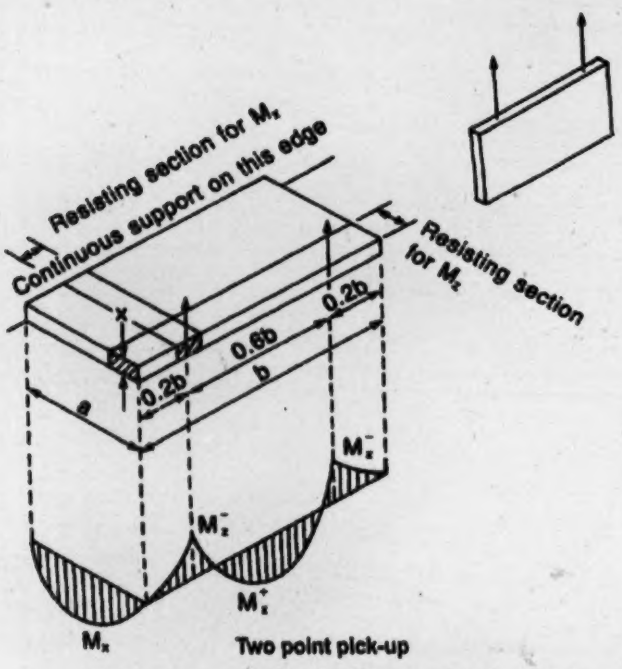
$w_f = 1.5$ kPa x 2.385 m = 3.58 kN/m

$$M_f = \frac{w_f \times L^2}{8} = \frac{3.58 \times 9.144^2}{8} = 37.39kN \cdot m$$

Note: Ignore benefit of parapet

Stripping and Handling:

Try a 2 point edge pick (The following figure is from the CPCI design manual)



Two point pick-up

Maximum moments (approximate)

$M_x = 0.0625\ wa^2b$

$M_z^- = M_z^+ = 0.0125\ wab^2$

M_x acting over a section of width $\frac{a}{2}$, 10t or $\frac{b}{2}$, whichever is less

M_z acting over a section of width 0.3b, 10t or $\frac{a}{2}$, whichever is less

Stripping equivalent static multiplier = 1.3 Governs

Yard handling and erection static multiplier = 1.2

Note: Panels are stored, shipped, and handled on edge. Erection is accomplished with panel rotated in the air. Thus, stripping governs over handling and erection.

Assume self weight = 23 kN/m^3 x (0.0762 x 2) m = 3.51 kPa

w = 1.3 x 3.51 kPa = 4.56 kPa

$$M_x = 0.0625wa^2b = 0.0625 \times 4.56 \times 2.385^2 \times 9.754 = 15.81kN \cdot m$$

M_x is resisted by an effective width of:

a/2 = 2385/2 = 1192.5 mm Governs
10t = 10 (76.2 + 101.6 + 76.2) = 2540 mm
b/2 = 9754/2 = 4877 mm

$$M_z^- = M_z^+ = 0.0125wab^2 = 0.0125 \times 4.56 \times 2.385 \times 9.754^2 = 12.93kN \cdot m$$

M_z is resisted by an effective width of:

0.3b = 0.3 (9754) = 2926.2 mm
10t = 10 (76.2 +101.6 +76.2) = 2540 mm
a/2 = 2385/2 = 1192.5 mm Governs

CHECK PANEL UNDER STRIPPING LOADS

For stripping, handling and erection, assume 80% composite action

$$I = 0.074\times10^9 + 0.8\times\left(1.278\times10^9 - 0.074\times10^9\right) = 1.037\times10^9\ mm^4/m$$

$$S = \frac{I}{\bar{y}} = \frac{1.037\times10^9}{127.0} = 8.165\times10^6\ mm^3/m$$

Transverse Direction

Allowable tensile stress in concrete: $f_{c(tensile)} = 0.6\sqrt{f_{ci}'} = 0.6\sqrt{20} = 2.68MPa$

$$f_c = \frac{M_x}{S_x} = \frac{15.81kN\cdot m\times10^6}{(8.165\times10^6\ mm^3/m)\times1.1925m} = 1.62MPa < 2.68 \text{ MPa therefore ok}$$

Note: Provide a minimum reinforcement ratio of 0.001 A_g. Shrinkage and temperature steel is not required for one-way precast / prestressed panels not wider than 12 ft. (10M spaced up to approx. 1350 mm both wythes)

Longitudinal Direction

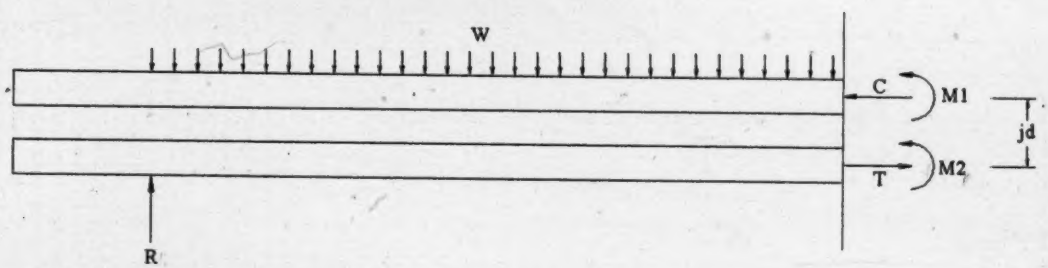
Immediately after transfer the average prestress force in the concrete is

$$f_{p/s} = \frac{0.68\times1860MPa\times219.2mm^2}{2385mm\times76.2mm} = 1.53MPa$$

Allowable tensile stress in concrete: $f_{c(tensile)} = 1.53 + 0.6\sqrt{20} = 4.21MPa$

$$f_c = \frac{M_z}{S_z} = \frac{12.93kN\cdot m\times10^6}{(8.165\times10^6\ mm^3/m)\times1.1925m} = 1.32MPa < 4.21 \text{ MPa therefore ok}$$

CHECK PANEL UNDER FACTORED WIND LOADS



$$M_r = M_1 + M_2 + V\times jd$$

where V is the lesser of ($\phi_{sc}V_{sc}$) or ($\phi_p A_p f_{pr}$ for tension wythe)

Moment transferred through shear connectors

Try 12 steel 3/4" x #13 Standard expanded metal connectors (therefore 6 connectors on each half of the panel)

ϕ_{sc} x V_{sc} = 6 x 16.19 = 97.11 kN

$$M_{sc} = \phi_{sc} \times V_{sc} \times jd$$

$$jd = \frac{h_{comp}}{2} + h_{insul} + \frac{h_{tens}}{2} = \frac{76.2}{2} + 101.6 + \frac{76.2}{2} = 177.8mm$$

$$M_{sc} = 97.11 \times 177.8 \times 10^{-3} = 17.27kN \cdot m$$

Determine M_1 – Moment Resistance of compression wythe

Uncracked Transformed Section Properties

$$n = \frac{E_p}{E_c} = \frac{190{,}000}{23664} = 8.03$$

$$A_t = b \times h_{comp} + (n-1) \times A_{p(comp)} = 1.833 \times 10^5 mm^2$$

$$y_t = \frac{\sum (A \times \tilde{y})}{\sum A} = \frac{2385 \times 76.2 \times 38.1 + (8.03-1) \times 219.2 \times 38.1}{2385 \times 76.2 + (8.03-1) \times 219.2} = 38.1mm$$

from extreme tension fibre

$$e = y_t + x_1 - h_{comp} = 38.1 + 38.1 - 76.2 = 0mm$$

$$I = \sum \left(\frac{b \times h^3}{12} + A \times d^2 \right) = 8.794 \times 10^7 mm^4$$

$$S = \frac{I}{y_t} = \frac{8.794 \times 10^7}{38.1} = 2.308 \times 10^6 mm^3$$

Determine M_1

Note: The compressive force resulting from shear transfer through the shear connectors between the two wythes is not considered when calculating M_1. The increased capacity that results from the compression force in the compression wythe is approximately equal to the decrease in capacity experienced by the tension wythe when incorporating the tensile force.

Note: It was seen from the Predicted Behaviour Example (Appendix A) that the concrete wythes do not fully develop their individual plastic moment resistance prior to the shear connectors failing. As a result M_1 is limited to M_{cr} for simplicity.

$$M_{cr} = \left(\frac{P_e}{A_t} + \frac{P_e \times e}{S} + 0.6\lambda\sqrt{f_c'}\right) \times S \times 10^{-6}$$

$$= \left(\frac{1116 \times 219.2}{1.833 \times 10^5} + \frac{1116 \times 219.2 \times 0}{2.308 \times 10^6} + 0.6 \times 1 \times \sqrt{35}\right) \times 2.308 = 11.27 kN \cdot m$$

Determine M_2 – Moment Resistance of tension wythe

Transformed Section Properties

$$n = \frac{E_p}{E_c} = \frac{190{,}000}{23664} = 8.03$$

$$A_t = b \times h_{tens} + (n-1) \times A_{p(tens)} = 1.833 \times 10^5 mm^2$$

$$y_t = \frac{\sum(A \times \tilde{y})}{\sum A} = \frac{2385 \times 76.2 \times 38.1 + (8.03-1) \times 219.2 \times 38.1}{2385 \times 76.2 + (8.03-1) \times 219.2} = 38.1 mm$$

from extreme tension fibre

$$e = y_t - x_2 = 38.1 - 38.1 = 0 mm$$

$$I = \sum\left(\frac{b \times h^3}{12} + A \times d^2\right) = 8.794 \times 10^7 mm^4$$

$$S = \frac{I}{y_t} = \frac{8.794 \times 10^7}{38.1} = 2.308 \times 10^6 mm^3$$

Determine M_2

Note: The tensile force resulting from shear transfer through the shear connectors between the two wythes is not considered when calculating M_2. The decreased capacity that results from the tension force in the tension wythe is approximately equal to the increase in capacity experienced by the compression wythe when incorporating the compression force.

Note: It was seen from the Predicted Behaviour Example (Appendix A) that the concrete wythes do not fully develop their individual plastic moment resistance prior to the shear connectors failing. As a result M_2 is limited to M_{cr} for simplicity.

$$M_{cr} = \left(\frac{P_e}{A_t} + \frac{P_e \times e}{S} + 0.6\lambda\sqrt{f_c'}\right) \times S \times 10^{-6}$$

$$= \left(\frac{1116 \times 219.2}{1.833 \times 10^5} + \frac{1116 \times 219.2 \times 0}{2.308 \times 10^6} + 0.6 \times 1 \times \sqrt{35}\right) \times 2.308 = 11.27 kN \cdot m$$

Ultimate Bending Moment Capacity

$M_r = 11.27 + 11.27 + 17.27 = 39.81 kN \cdot m > 37.37$ kN·m therefore ok

$w_r = 1.60 kPa$

% Composite Action at ultimate limit state

$\%Composite = \frac{\phi_{sc} \times V_{sc}}{T_2} \times 100\%$ but not greater than 100%

Assume Fully Composite Panel

$C = \phi_c \alpha_1 f_c' ba = 0.65 \times 0.798 \times 35 \times 2385 \times (0.883c) \times 10^{-3} = 38.233c$

$T = \phi_p A_p f_{pr}$

assume $f_{pr} = f_{pu}\left(1 - k_p \, c/d_p\right)$

$T_1 = 0.9 \times 219.2 \times 1860 \times \left(1 - 0.28 \times c/38.1\right) \times 10^{-3} = 366.94(1 - 0.007349c)$

$T_2 = 0.9 \times 219.2 \times 1860 \times \left(1 - 0.28 \times c/215.9\right) \times 10^{-3} = 366.94(1 - 0.001297c)$

from $C = T_1 + T_2$ solve for c

$c = 17.724 mm$

therefore $T_2 = 358.50$ kN

$$\%Composite = \frac{97.11}{358.50} \times 100\% = 27.1\%$$

CHECK PANEL UNDER SERVICE LOADS

The panel is 27.1% composite under service conditions

$I = 0.074 \times 10^9 + 0.271 \times \left(1.278 \times 10^9 - 0.074 \times 10^9\right) = 0.400 \times 10^9 \, mm^4 / m$

$$S = \frac{I}{\bar{y}} = \frac{0.400 \times 10^9}{127.0} = 3.150 \times 10^6 \, mm^3 / m$$

Longitudinal Direction

Under service load conditions the average prestress force in the concrete is

$$f_{p/s} = \frac{0.60 \times 1860 MPa \times 219.2 mm^2}{2385 mm \times 76.2 mm} = 1.35 MPa$$

Allowable tensile stress in concrete: $f_{c(tensile)} = 1.35 + 0.6\sqrt{35} = 4.90 MPa$

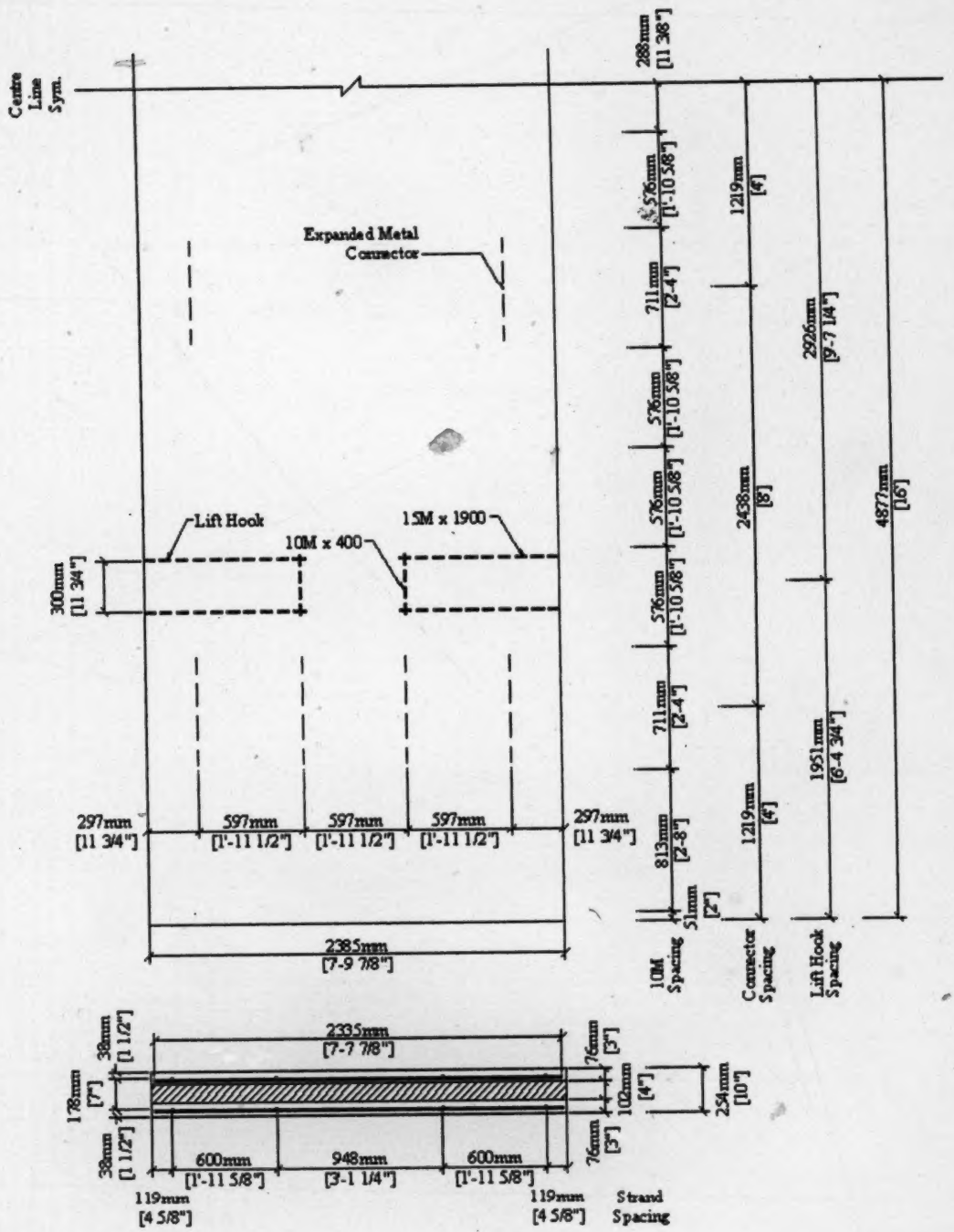
Bending moment under service load conditions:

$$M = \frac{(1.0KPa \times 2.385m) \times 9.144^2}{8} = 24.93KN \cdot m$$

$$f_c = \frac{M}{S} = \frac{24.93KN \cdot m \times 10^6}{(3.150 \times 10^6 mm^3/m) \times 2.385m} = 3.32MPa < 4.90 \text{ MPa therefore ok}$$

DESIGN SUMMARY

Concrete Stripping Strength:	20 MPa
28-day Concrete Strength:	35 MPa
Prestressing Strand:	4 – 3/8" Grade 1860 Low Relaxation Strands per wythe Stressed to 0.70 f_{pu}
Transverse Steel:	10M spaced up to 1350 mm in each wythe
Shear Connectors:	12 – Galvanized 3/4"-#13 Standard Expanded Metal Connectors
Insulation:	DOW Panelmate Insulation


Centre Line Sym.
288mm [11 3/8"]
Expanded Metal Connector
Lift Hook
10M x 400
15M x 1900
300mm [11 3/4"]
297mm [11 3/4"]
597mm [1'-11 1/2"]
597mm [1'-11 1/2"]
597mm [1'-11 1/2"]
297mm [11 3/4"]
2385mm [7'-9 7/8"]
576mm [1'-10 5/8"]
711mm [2'-4"]
576mm [1'-10 5/8"]
576mm [1'-10 5/8"]
576mm [1'-10 5/8"]
711mm [2'-4"]
813mm [2'-8"]
51mm [2"]
1219mm [4']
2438mm [8']
1219mm [4']
2926mm [9'-7 1/4"]
1951mm [6'-4 3/4"]
4877mm [16']
10M Spacing
Connector Spacing
Lift Hook Spacing
2335mm [7'-7 7/8"]
38mm [1 1/2"]
178mm [7"]
38mm [1 1/2"]
76mm [3"]
102mm [4"]
76mm [3"]
254mm [10"]
600mm [1'-11 5/8"]
948mm [3'-1 1/4"]
600mm [1'-11 5/8"]
119mm [4 5/8"]
119mm [4 5/8"]
Strand Spacing